VERIFICATION OF THE INLET CAPACITIES OF MODIFIED STORMWATER KERB INLETS AND THE DEVELOPMENT OF NEW DESIGN CURVES

BY

PIETER GROBLER

THESIS COMPLETED IN THE PARTIAL FULFILMENT OF THE REQUIREMENTS FOR THE DEGREE

MEng(Civ)

AT THE UNIVERSITY OF STELLENBOSCH

PROFESSOR A ROOSEBOOM Pr Eng

SUPERVISOR

STELLENBOSCH MARCH 1994

DECLARATION

I, the undersigned, hereby certify that the work included in this thesis is my own original work and has not previously been submitted either wholly or in part to any other university for the acquisition of a degree.

Signed: ..

Date: 1994-02-21

AKNOWLEDGEMENTS

To complete a project of this kind is virtually impossible without the support of colleagues, friends and family. I hereby wish to express my sincere gratitude to each and everyone who contributed to this thesis, with special reference to the following people:

- Professor A Rooseboom, my supervisor, for his guidance, support and patience throughout the research project, especially during those difficult periods in which little progress were made.

- Mr R Corin, my former superior, who initiated this research project and made valuable contributions to the final product.

- My colleagues at the City Engineer's Department for their support and practical inputs.

- Messrs L W Fourie and J A le Roux for their contribution in general but especially for their inputs towards the format and appearance of the new design curves.

- My parents, who supported me from day one of my studies and throughout this research project.

- My wife Ilse, for her support and sacrifices that she had to make during our relationship and marriage.

- To my Creator, without whom nothing is possible.

SYNOPSIS

Various aspects affecting the inlet capacities of modified stormwater kerb inlets were investigated. Investigations centred on the influence of effective inlet length, ratios of upstream transition section length to inlet section length and road crossfall. The suitability of existing design curves for use in the design of modified kerb inlets was verified and new more "user-friendly" design curves were developed. Attempts were made to develop a sound theoretical model to predict inlet capacities for stormwater kerb inlets as the lack of such a model had previously been identified as a major shortcoming in the design of stormwater kerb inlets.

A detailed study of relevant literature and of various full scale model test results confirmed the fact that the effective inlet length is the most important variable affecting the inlet capacity of stormwater kerb inlets. Model studies on various combinations of transition section lengths and inlet section lengths have indicated that in the case of supercritical flow the inlet capacity of stormwater kerb inlets is not sensitive to changes in the ratio between the inlet section length and the upstream transition section length. Furthermore no marked effect on the inlet capacity of a modified stormwater kerb inlet was detected when the road crossfall was increased from 2% to 3%. The inlet capacities predicted by the existing design curves were found to be consistent, although on the conservative side, when compared to the inlet capacities obtained from the full scale model tests. New design curves derived from the existing design curves were developed to provide a less cumbersome procedure in the design of stormwater kerb inlets. The development of an improved comprehensive theoretical model based on pure hydraulic principles was not possible due to the extremely complex nature of flow conditions at stormwater kerb inlets.

The most important findings emanating from this research project are:

- The confirmation of the "effective inlet length" concept, whereby a part of the expensive inlet section is replaced by an equivalent length of inexpensive

transition section upstream of the kerb inlet, which does not affect the inlet capacity of the kerb inlet negatively and thereby results in a substantial saving in the cost of stormwater kerb inlets.

- Model tests revealed that for supercritical flows the upstream transition section can be made up to 6 times longer than the inlet section with a maximum length of 6m, without any negative effect on the inlet capacity of the kerb inlet. Similarly an analysis of previous research results has indicated that even in cases of subcritical flow the upstream transition section can be made twice the length of the inlet section.

- The full scale model tests also indicated that an upward adjustment of 30% in the inlet capacities as predicted by the existing design curves was justified. The modified design curves proved to be suitable for the design of conventional stormwater kerb inlets as well as for modified stormwater kerb inlets where part of the inlet section is replaced by a transition section. A new set of "easy to use" design curves was developed specifically for urban applications.

- Guidelines for determining upstream transition section lengths were drawn up for use in conjunction with the existing and new design curves for the design of stormwater kerb inlets.

SAMEVATTING

Verskeie aspekte wat die inlaatkapasiteit van verbeterde randsteeninlate beïnvloed is ondersoek. Die belangrikste aspekte wat ondersoek is, was die invloed van effektiewe inlaatlengte en die moontlike verhouding van die stroomop oorgangslengte tot die werklike inlaatlengte sowel as dwarshelling. Die ontwerpgrafieke wat tans gebruik word om randsteeninlaatlengtes te bepaal is ook ondersoek aan die hand van modeltoetse op volskaalse modelle om die grafieke se akkuraatheid te toets. Meer gebruikersvriendelike ontwerpgrafieke is ook ontwikkel. 'n Belangrike tekortkoming tot op datum, naamlik 'n geskikte teoretiese model om die inlaatkapasiteit van randsteeninlate te beskryf, is aangespreek. 'n Voorvereiste vir so 'n model was dat die model gebaseer moes word op suiwer hidrouliese beginsels.

Die bestudering van vorige navorsing asook die uitvoering van verskeie modelstudies op volskaalse randsteeninlate het die effektiewe inlaatlengte as die belangrikste veranderlike wat die inlaatkapasiteit van randsteeninlate bepaal geïdentifiseer. Tydens toetse uitgevoer op volskaalse modelle onder superkritiese vloeitoestande met verskillende kombinasies van oorgangslengtes en inlaatlengtes, vir die dieselfde totale lengte het dit geblyk dat die inlaatkapasiteit grootliks onafhanklik is van die verhouding van die oorgangslengte tot die inlaatlengte. Uit die modeltoetse het dit ook geblyk dat paddwarsval 'n baie beperkte invloed op die inlaatkapasiteit van ransteeninlate gehad het toe die paddwarsval verander is van 2% na 3%. Die resultate wat verkry is uit die modeltoetse het ook aangetoon dat die bestaande ontwerpgrafieke konserwatief is in hul voorspelling van inlaatkapasieit van randsteeninlate. Die bestaande ontwerpgrafieke is voorts gebruik om nuwe meer gebruikersvriendelike ontwerpgrafieke te ontwikkel. Die ontwikkeling van 'n teoretiese model om inlaatkapasiteite volledig te voorspel was egter nie moontlik nie vanweë die uiters komplekse aard van die vloei by randsteeninlate.

Die belangrikste bevindinge van hierdie studie kan soos volg opgesom word:

- Afdoende bewyse is gevind dat die "effektiewe inlaatlengte" konsep, waarvolgens 'n gedeelte van die duur inlaatgedeelte met 'n goedkoper oorgangsstuk van dieselfde lengte vervang kan word sonder om inlaatkapasiteit in te boet, wel suksesvol aangewend kan word om kostes in die ontwerp van stormwater randsteeninlate te bespaar.

- Die modelstudies het aangetoon dat die oorgangstuk tot 6 keer langer as die inlaatgedeelte mag wees tydens superkritiese vloeitoestande sonder enige negatiewe invloed op die inlaatkapasiteit van die randsteeninlaat. Voorts kon daar vanaf vorige navorsingswerk afgelei word dat selfs vir subkritiese vloei die oorgangstuk tot 2 keer langer as die inlaatgedeelte mag wees.

- Modeltoetse op volskaalse modelle het verder aangetoon dat die bestaande ontwerpgrafieke met 30% opwaarts aangepas kan word in die raming van inlaatkapasiteite van randsteeninlate. Die aangepaste bestaande ontwerpgrafieke is geskik om gebruik te word in die ontwerp van konvensionele randsteeninlate sowel as in die ontwerp van gemodifiseerde randsteeninlate waar 'n deel van die inlaatgedeelte vervang word met 'n oorgangstuk. Daar is ook voortgegaan met die ontwikkeling van gebruikersvriendelike ontwerpgrafieke vir munisipale gebruik.

- Riglyne is ook ontwikkel vir die bepaling van stroomop oorgangslengtes. Diè riglyne kan saam met die bestaande en die nuwe ontwerpgrafieke gebruik word by die ontwerp van stormwater randsteeninlate.

CONTENTS

LIST OF TABLES

LIST OF FIGURES

SYMBOLS AND TERMINOLOGY

Terminology

Effective inlet length	- The combined length contributing to the inlet capacity of the kerb inlet. The effective inlet length may comprise an upstream transition section, an inlet section and a downstream transition section.
SARB	- South African Road Board
Existing design curves	- Design curves currently used as published in the Road Drainage Manual of the South African Road Board
New design curves	- Design curves developed as a result of this research project

Symbols

Q_0	- Flow approaching kerb inlet
Q	- Inlet capacity of kerb inlet
L	- Inlet length of kerb inlet
L_1	- Length of depressed gutter upstream of kerb inlet
L_2	- Length of depressed gutter downstream of kerb inlet
g	- Gravitational acceleration
q	- Carry-over flow
Θ	- Road crossfall
v	- Velocity of the approaching flow for undepressed kerb inlet
v_0	- Velocity of the approaching flow for depressed kerb inlet

y	- Flow depth against kerb for undepressed kerb inlet
y_0	- Flow depth against kerb for depressed kerb inlet
s	- Road gradient
a	- Depth of depressed gutter
z	- $z = s.L_1 + a$
F	- Froude number
C	- Constant
K	- Constant
b	- Depth of non-standard depressed gutter
a'	- $b-L_2.s$ / 1-4s
dQ	- Disharge per unit length
dx	- Unit length
y_L	- Depth of flow along the kerb inlet ($y = y_S(1-x/L)$
y_S	- Flow depth at upstream end of kerb inlet
L'	- Kerb inlet length required for partial interception
Q'	- Partially intercepted flow by inlet length L'
m	- Discharge coefficient depending on approach velocity of flow
dv/dt	- Acceleration of element
ΣF	- Sum of the unbalanced forces
F_{1X}	- Force due to the weight component
F_{2X}	- Unbalanced force due to the pressure difference across the width of the element
γ	- Specific weight of water
dv_X/dt	- Acceleration of element in the x-direction
ρ	- density of water
R	- Hydraulic radius
k	- Absolute roughness parameter
Fr	- Froude number

CHAPTER 1: INTRODUCTION

Stormwater kerb inlets intercept stormwater from the surface of the road and discharge it into a stormwater drainage system which can be either an open system or an underground system.

A stormwater kerb inlet has been defined as follows: "A curb inlet is a vertical opening through which the gutter flow passes." (The John Hopkins University, 1956). As crude as this may sound, this basic definition still describes the basic function of the stormwater kerb inlet.

Hydraulically speaking, kerb inlets are not the most effective way of intercepting streetflow as they are placed parallel to the oncoming flow direction. Nevertheless, they are widely used for their practical advantages, such as relatively low construction and maintenance costs as well as road safety considerations.

Through the years, a considerable amount of time and money has been spent on research into the field of stormwater management. This has lead to well-designed and cost effective systems to drain stormwater, once the water was intercepted. An area which was neglected was the mechanism to intercept the water, with special reference to kerb inlets. The aim of this research is to verify existing design curves, the effective inlet length concept and to develop new, user-friendly, design curves.

A kerb inlet is in most cases literally regarded as a vertical opening in the kerb face. This approach necessarily leads to hydraulic ineffective and therefore expensive kerb inlets. The cost of this type of inlet varies from about R2 500 for a three metre inlet to about R4 200 for a six metre inlet (1994 prices).

Further research carried out indicated that part of the expensive inlet length can be replaced by a cheaper transition length without affecting the inlet capacity adversely (Zwamborn, 1966; Forbes, 1976 and Rooseboom, 1988). This is especially true for supercritical flow which, as will be shown later, is the most common flow type for flow of stormwater along roads. It is well known that supercritical flow cannot change direction rapidly and this makes the upstream portion of the inlet ineffective. If the ineffective upstream portion of the inlet is replaced with a transition section, the cost of a kerb inlet can be reduced by a considerable amount. This is obvious if the cost of about R250 for a three metre transition section is compared with R2 500 for a three metre kerb inlet.

The following questions have arisen:

1.1 Are the existing design curves both accurate and suitable for use as design curves for modified stormwater kerb inlets?

1.2 Is the inlet capacity of the kerb inlet negatively affected, should a part of the inlet be replaced by a transition section?

1.3 Does the gradient of the road place any restrictions on the length of the transition?

1.4 Is it possible to develop an accurate, theoretical model in order to determine the inlet capacity of a kerb inlet?

The aim in the following chapters is to find answers to these questions, commencing with background information and recent developments in the design of stormwater kerb inlets.

CHAPTER 2: EARLIER RESEARCH ON THE HYDRAULIC BEHAVIOUR OF STORMWATER AT KERB INLETS

In order to embark on a project of this nature, a study of the major developments in stormwater kerb inlet design over the last four decades has been necessary.

2.1 The John Hopkins University (1956)

In the early fifties the Institute of Co-operative Research of The John Hopkins University conducted a research study into the hydraulic behaviour of stormwater kerb inlets. All work was conducted under the direction of the Sanitary Engineering Department of the university in co-operation with the Department of Civil Engineering. The results of the study were published in a series of articles. In this research project only the work on kerb inlets as published will be referred to.

The series of model tests were all conducted on a wooden model, approximately 6.1 metres long and 0.9 metres wide. The gradient as well as the crossfall of the model could be altered in order to accommodate different gradients (longitudinal) and crossfall slopes. The study further included tests of kerb inlets under both undepressed and depressed conditions. The model tests were correlated with selective test results on prototype kerb inlets. These tests were carried out only where all the approaching water could be intercepted due to the unavailability of suitable gauging equipment.

The next step in the study was to develop a mathematical model to determine the inlet capacity of kerb inlets. This model was calibrated by using the results from the above-mentioned model tests. A brief discussion on the development of the mathematical model for both the undepressed and the depressed kerb inlets is included here.

2.1.1 Undepressed kerb inlets

The discharge through a stormwater kerb inlet depends mainly on the flow depth in the street and the length of the kerb inlet. In the case of the undepressed kerb inlet the crest depth is equal to the depth of kerb flow at the upstream end of the kerb inlet. The progressive decrease in the depth of kerb flow induces flow across the road. In the case of steep roads with high water velocities, the momentum in the longitudinal direction becomes dominant, with the result that the efficiency of the kerb inlet decreases.

A shortened version of the mathematical model for the undepressed kerb inlet as developed by The John Hopkins University is as follows:

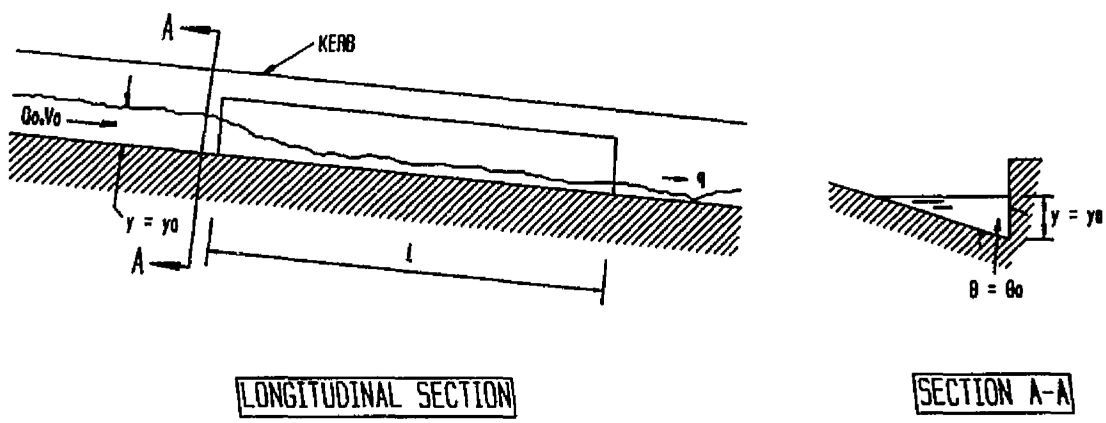


FIGURE 2.1.1: STORMWATER KERB INLET WITHOUT A DEPRESSED GUTTER

The inlet capacity, (Q), of a kerb inlet depends on the inlet length, (L), the gravitational acceleration, (g), the carry-over flow, (q), the crossfall of the road, (Θ), and the velocity, (v), of the approaching flow. As Q, L, g, q, v, y and Θ can be expressed in terms of two fundamental physical dimensions namely time and length, there can be five independent dimensionless parameters only:

$$Q/[L.y.(g.y)^{0.5}],\ v/(g.y)^{0.5},\ q/Q,\ \Theta \text{ and } (Q + q)/(v.y^2.\tan\Theta)$$

Manipulation of the above for a triangular cross-section leads to the following basic relationship:

$$Q/L.y.(g.y)^{0.5} = f\,[v/(g.y)^{0.5},\ q/Q,\ \Theta]$$

Model tests were carried out with the following ranges of variation in variables:

$v/(g.y)^{0.5}$: 1 - 3

q/Q : 0 - 0.7

$\tan\Theta$: 12, 24 and 48

The data from these tests indicated that:

$$Q/[L.y.(g.y)^{0.5}] = Q/[L.y_0.(g.y_0)^{0.5}] = K \qquad (2.1.1)$$

where K depends on Θ only. For $\tan\Theta$ (= $\tan\Theta_0$) = 12, 24 and 48, K = 0.23, 0.20 and 0.20 respectively.

Using Manning's formulae for y_0 and v_0, Equation 2.1.1 can be written as:

$$Q/L = 8.17K\,[(1+\sec\Theta_0)^{3/8}/(\tan\Theta_0^{15/16})].(Q_0/(s/n)^{0.5})^{9/16} \qquad (2.1.2)$$

Using Equation 2.1.2 the inlet capacity of an undepressed kerb inlet can be determined for a given inlet length, crossfall, Manning roughness coefficient and road gradient.

2.1.2 Depressed stormwater kerb inlets (Figure 2.1.2)

The general feeling at that stage at The John Hopkins University was that a depressed gutter in front of a kerb inlet would lead to an increase in inlet capacity.

In tests conducted, a depressed gutter of 63mm at a kerb inlet placed along a road with a gradient of 6.5%, increased the inlet capacity by a factor 10. The lengthening of the depressed gutter in an upstream direction from the kerb inlet, lead to very interesting results. The initial increase in length lead to an increase in inlet capacity, but beyond a certain length a decrease in inlet capacity was detected. The conclusion drawn from this was that the length of the depressed gutter was a function of the road gradient as well as the road crossfall. The research team recommended that this be investigated further at a later stage.

The research team of The John Hopkins University continued with their efforts to effectively increase the inlet capacity of a kerb inlet using a depressed gutter. Their next improvement was a triangular depressed gutter with the base of the triangle opposite the upstream end of the kerb inlet and the top of the triangle opposite the downstream end of the kerb inlet. This means that the crossfall in the depressed gutter increases in a downstream direction. With a depressed gutter that varies in width it means that the depth and crossfall of the depressed gutter is less pronounced in the road where traffic passes over the gutter but more pronounced at the downstream end of the kerb inlet where the depressed gutter does not interfere with the traffic. Comparative tests between the triangular-shaped depressed gutter and the constant-width depressed gutter indicated an increase in inlet capacity of up to 80% in the case of the triangular-shaped gutter.

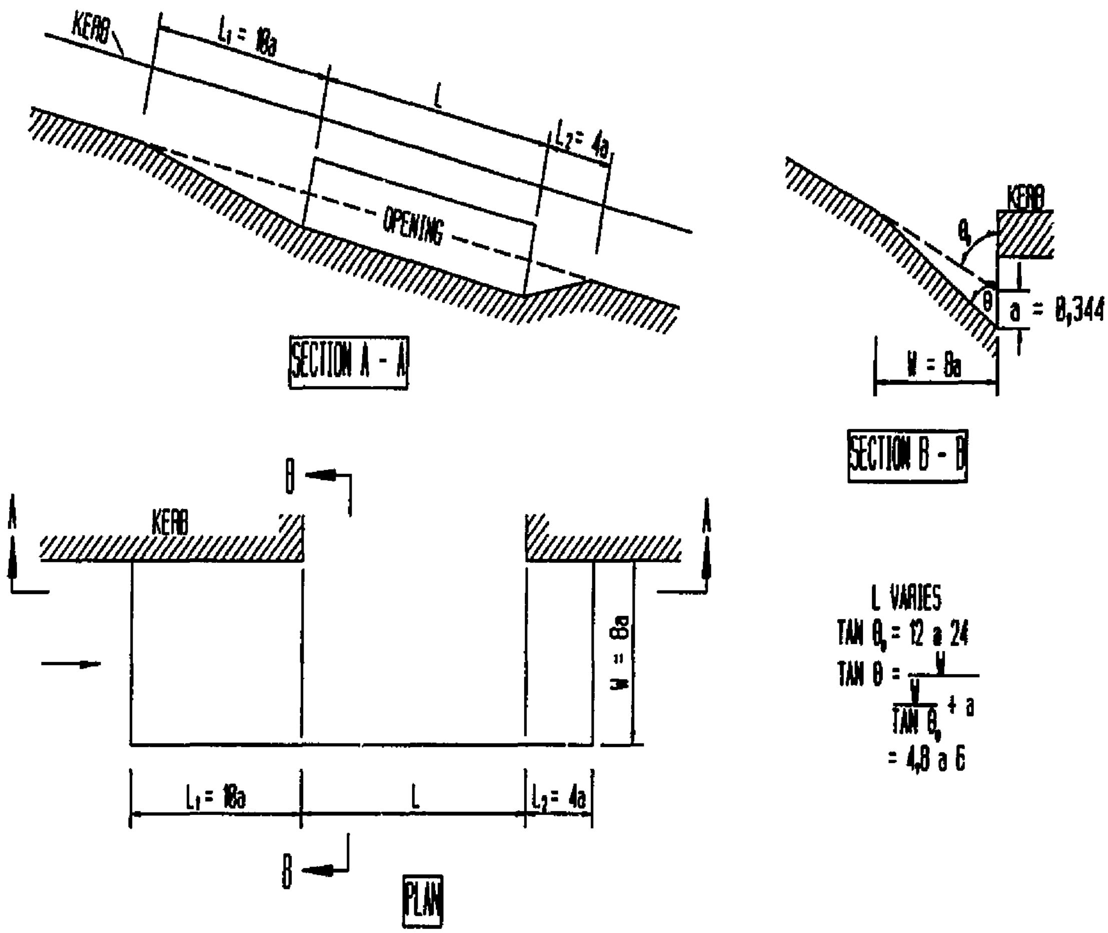


FIGURE 2.1.2: STORMWATER KERB INLET WITH DEPRESSED GUTTER

The research team realised that the mathematical model developed for the undepressed kerb inlet would not be suitable to determine the inlet capacity of the depressed kerb inlet, for the following reasons:

- In the case of the undepressed kerb inlet $y = y_0$. In the case of the depressed kerb inlet this is not the case and therefore y_0 cannot be used in Equation 2.1.1

- The depression causes a damming effect at the downstream end of the kerb inlet. This induces a hydraulic jump which increases the inlet capacity of the kerb inlet.

To develop a mathematical model for a depressed kerb inlet, one important assumption was necessary. This was that the depression extends far enough in an upstream direction beyond the upstream end of the kerb inlet so as to allow the surface of the water to return to a horizontal condition before the beginning of the kerb inlet. This also simplified the calculation of y at the upstream end of the kerb inlet.

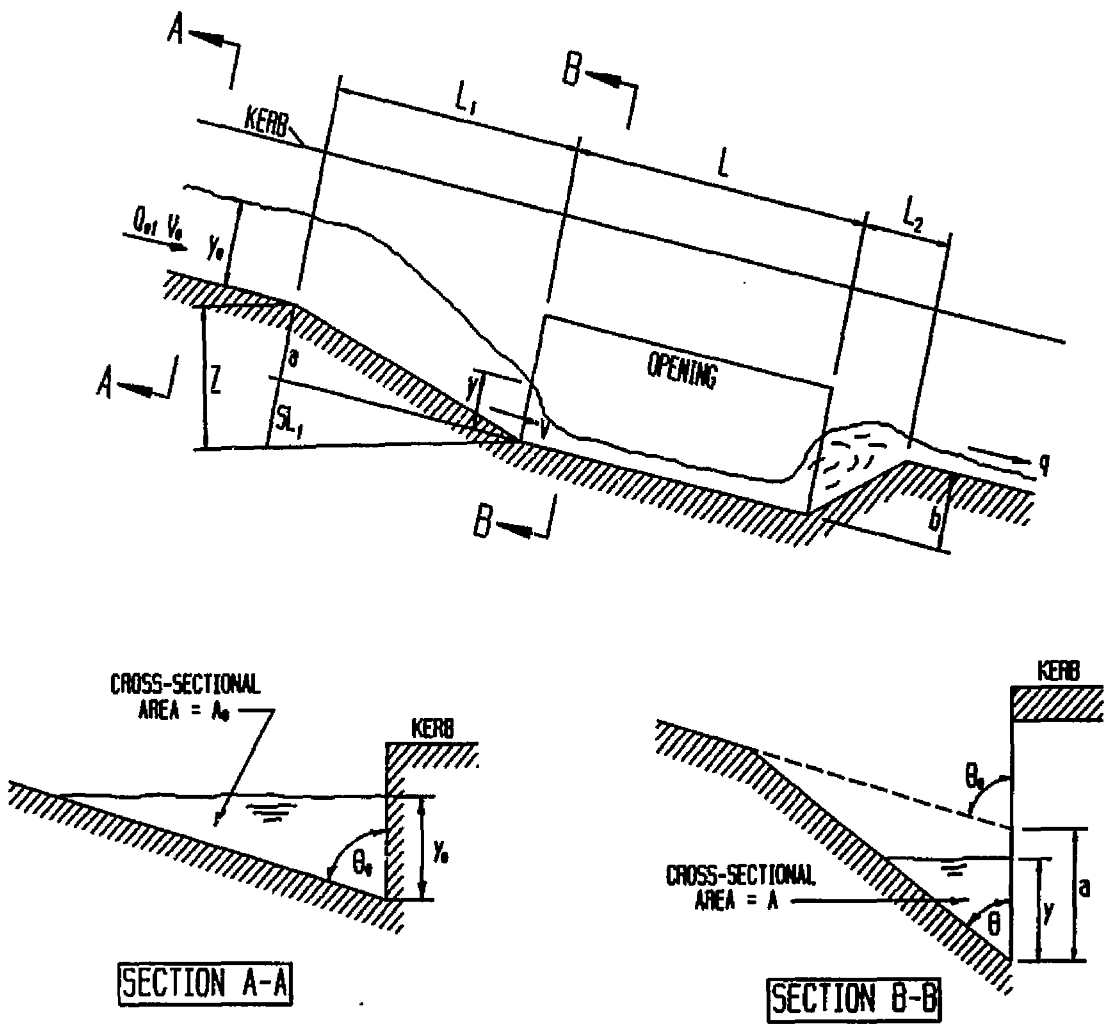


FIGURE 2.1.3: STORMWATER KERB INLET AND FLOW DIMENSIONS

Using the principle of energy conservation between cross-sections A-A and B-B in Figure 2.1.3:

$$v_0^2/2g + y_0 + z = v^2/2g + y + h_{fL1} \qquad (2.1.3)$$

Assuming that the energy gradient of the flow in the depressed gutter does not differ significantly from the energy gradient of the flow in the road, it can be said that the energy losses are equal to $s.L_1$. (Figure 2.1.3)

Since $z = s.L_1 + a$

$$v_0 = Q_0/A_0$$

and $v = Q_0/A$

Equation 2.1.3 can be transformed into:

$$Q_0^2/2gA_0^2 + y_0 + a = Q_0^2/2gA^2 + y \qquad (2.1.4)$$

Since y is directly related to A, y can be determined for given values of the depth of the depression, a, and flow, Q_0, using Equation 2.1.4.

With the flow depth, y, the average velocity, v, and the flow, Q_0, at the upstream end of the kerb inlet all known, the inlet capacity of the kerb inlet can be determined. The inlet capacity, Q, depends on the crossfall, Θ (or the average crossfall if the width of the approaching flow exceeds the width of the depressed gutter), the inlet length, L, the depth of the depressed gutter, a, the length of the depressed gutter downstream of the kerb inlet, L_2, the carry-over flow, q, and lastly the gravitational acceleration, g.

By dimensional analysis these ten quantities lead to eight dimensionless terms:

$$Q/Ly(gy)^{0.5},\ v^2/gy,\ L/a,\ \Theta,\ L_2/a,\ q/Q_0,\ Q + q/Q_0,\ Q_0/vy^2\tan\Theta$$

As the last two quantities are constants, the following relationships exist between the remaining six quantities.

$$Q/Ly(gy)^{0.5} = f\ (v^2/gy,\ L/a,\ \Theta,\ L_2/a,\ q/Q_0)$$

For convenience, Equation 2.1.1 can be modified for the depressed kerb inlet by introducing the term, C, to account for the increase in inlet capacity due to the backwater effect.

$$Q/Ly(gy)^{0.5} = K + C \qquad (2.1.5)$$

Whilst K has been found to be dependent on the crossfall only, it is obvious that C is dependent on a greater number of variables:

$$C = f\ (v^2/gy,\ L/a,\ \Theta,\ L_2,\ q/Q_0)$$

Further model tests were conducted to calibrate the value of C for a given crossfall.

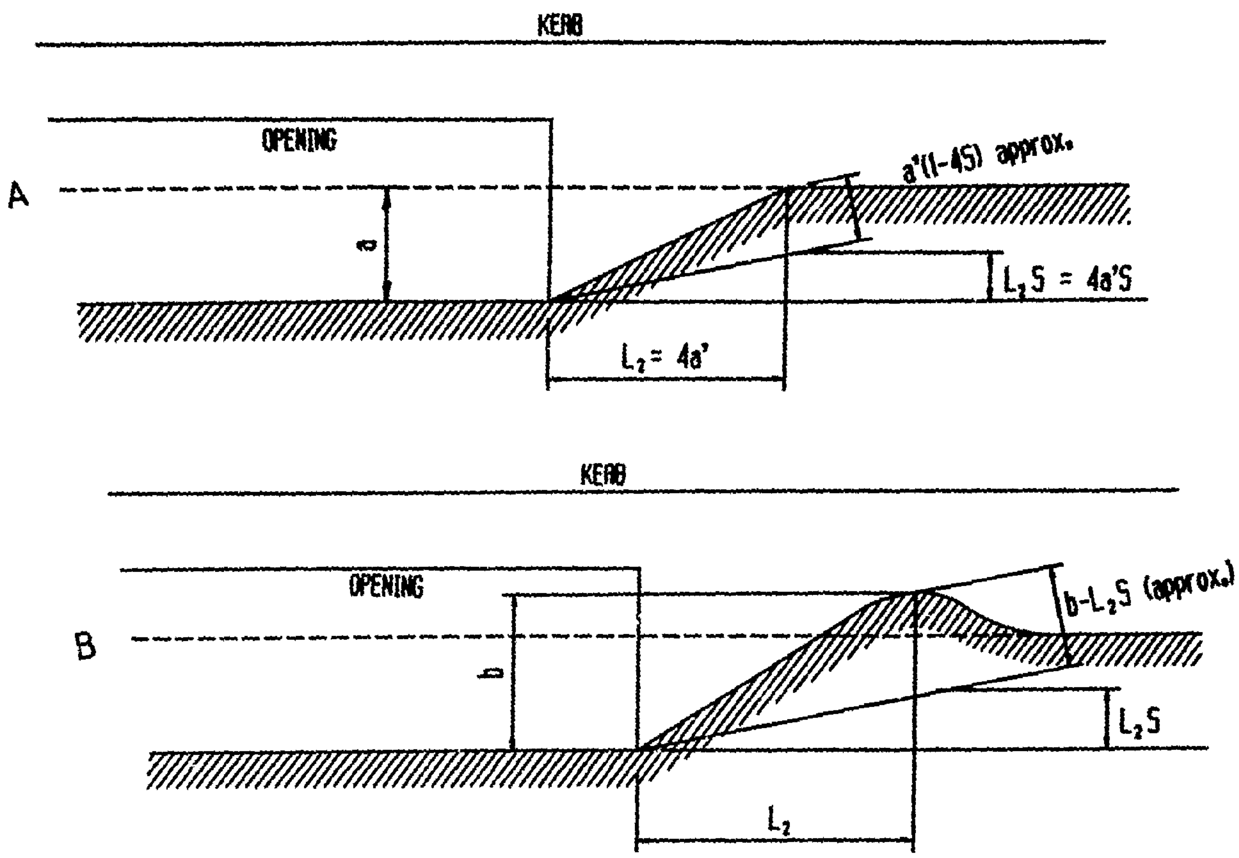


FIGURE 2.1.4: STANDARD AND NON-STANDARD DEPRESSED GUTTERS AT STORMWATER KERB INLETS

For a K-value of 0.23 the following equation for C was derived:

$$C = 0.45/1.12^M \tag{2.1.6}$$

Where $M = LF/a.\tan\Theta$

and $Fr = v^2/gy$

Equation 2.1.6 has been obtained with the length, L_2, of the depressed gutter downstream of the kerb inlet equal to 4 times the depth, a, of the depressed gutter. (Figure 2.1.4A). If the distance b (Figure 2.1.4B), is different from a, or where L_2/a is not equal to 4, it is recommended that Equation 2.1.6 be modified to:

$$C = 0.45/1.12^N \qquad (2.1.7)$$

Where $N = LF/a'.\tan\Theta$

and $a' = b - L_2s/1 - 4s$ in Figure 2.1.4

It can be seen that when a and b are equal to zero, C becomes zero and Equation 2.1.5 is identical to Equation 2.1.1. Equation 2.1.5 is, therefore, the general equation for the inlet capacity of stormwater kerb inlets and Equation 2.1.1 is a special form applicable to undepressed kerb inlets.

C can now be determined by using either Equation 2.1.6 or 2.1.7. For a given length, the inlet capacity of the kerb inlet can be determined using Equation 2.1.4.

The published findings of the research team at the John Hopkins University were as follows:

- With an increase in gradient the inlet capacity of a stormwater kerb inlet diminishes.

- An increase in road crossfall increases the inlet capacity of a stormwater kerb inlet.

- A depressed gutter concentrates the water and directs it into the stormwater kerb inlet, increasing the inlet capacity.

- The depth, length and the shape of the depressed gutter all have an influence on the inlet capacity of a stormwater kerb inlet.

- A long shallow depressed gutter is hydraulically just as effective as a short deep depressed gutter.

- A depression that decreases in width in a downstream direction is more effective than a constant width depression.

- The results obtained with the mathematical model compared reasonably well with the results from the test model.

2.2 Zwamborn (1966)

Although significant results were obtained at the John Hopkins University, they were mostly based on small scale model tests and prototype flows never exceeded 42.5l/s. In 1966 Mr J A Zwamborn, the then Head of the Hydromechanics Research Department, National Mechanical Engineering Research Institute of the C.S.I.R, conducted a research project into the inlet capacities of stormwater kerb inlets for use in South Africa.

To achieve his goal, tests were conducted on a full scale model as well as on a 1:6-scale model. The results obtained from the 1:6-scale model compared reasonably well with the results from previous model tests (The John Hopkins University, 1956). However, the results from the full scale model differed to a considerable degree, indicating the presence of scale effects. One of Zwamborn's conclusions was that the results obtained from small scale models should be carefully interpreted.

As it was difficult to manipulate the full scale model, it was decided to determine the effects of the more important variables in the full scale model and the effects of the variables of lesser importance in the smaller scale model.

In his theoretical approach Zwamborn attempted to satisfy the basics of open channel flow hydraulics and tried to minimise debatable assumptions. One of his major assumptions was that the depth of flow decreases linearly along the opening of an undepressed kerb inlet.

A brief description of his mathematical model highlights the most important assumptions and resulting equations (Zwamborn, 1966).

In the case of a linear decrease in the depth of flow along the opening of the kerb inlet the discharge per unit length is given by:

$$dQ = 3my_L^{3/2}dx \qquad (2.2.1)$$

with m : discharge coefficient depending on approach velocity of flow.

and $y_L = y_S(1 - x/L)$

Integration of Equation 2.2.1 yields:

$$Q/L = 1.2my_S^{3/2} \qquad (2.2.2)$$

y_S : depth of flow at the upstream end of the kerb inlet

The value Q/L, the inlet capacity, of the kerb inlet can thus be determined using Equation 2.2.2

To prove the validity of Equation 2.2.2, an extensive range of tests had to be carried out. To avoid the scale effects due to viscosity and surface tension it was decided to do as many tests as possible on the full scale model. Initially, all tests were performed with a crossfall of 2.5%. In the case of depressed kerb inlets the depth of the depression was set at 61mm. The influence of other crossfalls (2.0% and 3.33%) and different depression depths (31mm and 91mm) were later tested on the 1:6-scale model. According to Zwamborn, tests on both the full scale model as well as on the 1:6-scale model proved the validity of the assumption that the depth of flow decreases linearly along the opening of the kerb inlet.

Model tests on both models showed a sharp drop from y, which is the flow depth against the kerb upstream of the start of the inlet, to $y_{S,}$ the flow depth at the upstream end of the kerb inlet. Y is determined by using either the Manning or the Chezy equations. Plotting his results, Zwamborn eventually derived the following relationship between y and y_S:

$$y_0 = 0.53y^{0.83} \qquad (2.2.3)$$

Substituting Equation 2.2.3 into 2.2.2 yields:

$$Q/L = 0.46my^{1.25} \qquad (2.2.4)$$

In determining the inlet capacity, the results obtained from the full scale model agreed remarkably well with those results obtained from Equation 2.1.5 as developed by Zwamborn namely:

$$Q/L = 0.8y^{1.25} \qquad (2.2.5)$$

with $\qquad$ m : 1.7

According to Zwamborn, tests conducted on undepressed kerb inlets showed that the results obtained from the full scale model and the 1:6-scale model agreed fairly well, except at low flows. However, in the case of the depressed kerb inlet, the difference was more pronounced. These results again highlighted the risks associated with the use of small scale models. However, the advantage of a depressed gutter became obvious from the results.

To determine an optimum depression depth Zwamborn used the 1:6-scale model. The length of the depressed gutter extending upstream of the kerb inlet was fixed at 0.91m. From these results it was concluded that to increase the depth of the gutter beyond 61mm, did not increase the inlet capacity of the kerb inlet.

Further tests to determine the effect of the crossfall on the inlet capacity indicated that an increase from 2% to 3.33% in the crossfall did not increase the inlet capacity of the kerb inlet significantly.

At this stage Zwamborn concluded that a more economical solution to the problem would be to intercept only a portion of the approaching flow. By again using Equation 2.2.1 and the assumption that the flow depth decreases linearly along the opening of the kerb inlet, he derived the following equation for partially intercepted flow.

$$Q'/Q = 1 - (1 - L'/L)^{5/2} \qquad (2.2.6)$$

Where L : inlet length of kerb inlet

L' : kerb inlet length required for partial interception

and Q' : partially intercepted flow by inlet with length L'

Strictly speaking Equation 2.2.6 is only applicable in the undepressed case, but as Zwamborn indicated that a linear decrease in the flow depth prevails at depressed kerb inlets as well, he concluded that Equation 2.2.6 could be used for both the depressed and undepressed cases.

Zwamborn's findings can be summarised as follows:

- The optimum depth of the depression is 61mm.
- The road crossfall has no significant effect on the inlet capacity of a stormwater kerb inlet.
- Partial interception appeared to be an attractive economical option.
- An expression based on the basic weir formula and on linearly decreasing depth along the kerb inlet, proved to be essentially correct.

2.3 Forbes (1976)

In 1976 Mr H.J.C Forbes in conjunction with the Durban Municipality, identified a shortcoming in the available data on the inlet capacities of kerb inlets. Up to then all efforts had been concentrated on flatter road gradients. He decided to take the research a step further by doing tests on steep gradients. Forbes also decided to develop a numerical model, in order to determine the inlet capacity of kerb inlets, as previous efforts to develop a sound theoretical model based purely on descriptions of steady flow conditions upstream of the kerb inlet had not been successful.

This study will not include a detailed discussion of the development of the numerical method as a second discussion would only lead to the duplication of previous comments on the numerical method which are not relevant to this work.

In this study the author has decided to concentrate on the empirical results of the Forbes tests because of their importance to this research project.

The pilot numerical method produced results that underestimated the inlet length of kerb inlets. An empirical constant was applied to the model and this resulted in reasonable agreement with the results obtained from previously developed formulae (The John Hopkins University, 1956). The numerical method was then compared with the results from the full scale model tests conducted by Zwamborn. For gradients up to 2.5% the numerical method generally correlated well for both 100% and 80% interception but not for lower interception rates. On steeper gradients the numerical method predicted longer inlet lengths than the full scale model. This tendency appeared again when the numerical model was modified for depressed kerb inlets. It was noted by Forbes that the full scale model tests by Zwamborn were confined to gradients flatter than 2.5%. For steeper gradients Zwamborn used the 1:6-scale model. The 1:6-scale model produced longer inlet lengths than those predicted by the full scale model. At this stage Forbes felt confident that the numerical method was fairly accurate especially for flatter gradients. There was however no confirmation that the model would be accurate in the case of depressed kerb inlets on steep gradients.

The next step was to determine a theoretical optimum inlet length. The numerical method was again modified. The numerical model produced some surprising results. They were:

- That for steeper slopes, the optimum depressed gutter upstream of the kerb inlet exceeded the 0.91m length previously regarded as the optimum depressed gutter length, (Zwamborn, 1966) by a considerable amount.

- That the combination of depressed gutter length upstream of the kerb inlet and the actual kerb opening inlet length could be varied within wide limits without affecting the inlet capacity significantly.

The results obtained above prompted the Durban Municipality to carry out a range of model tests on a depressed kerb inlet with varying upstream depressed gutter lengths. For this purpose they used a 1:6-scale model with prototype dimensions for the depressed gutter of 305mm wide and 61mm deep. Tests were conducted on gradients between 1.9% and 12.1% and for flows ranging between 24l/s and 111l/s. Compared with the full scale model tests of Zwamborn and the numerical model, the model was consistently conservative, again an indication of the presence of scale effects in small models. It was interesting that a 20% downward adjustment in the required inlet lengths, of the Durban Municipality model, gave a good correlation with both the full scale model of Zwamborn and the numerical model of Forbes.

In each set of tests four or five combinations were tested with the constants being street gradient, street crossfall, total inlet length (kerb opening + upstream depression) and total streetflow. For tests within a set, the start of the kerb opening was varied with an increase in the upstream depressed gutter therefore leading to a decrease in the kerb opening. For each test the intercepted flow was expressed as a percentage of the constant streetflow. Typical results are presented in Table 2.3.1

TABLE 2.3.1: DURBAN MUNICIPALITY'S MODEL TESTS: EFFECT OF UPSTREAM DEPRESSED GUTTER ON INLET CAPACITY OF STORMWATER KERB INLETS

GRADIENT (%)	CROSS FALL (%)	TOTAL STREET-FLOW (l/s)	UPSTREAM GUTTER LENGTH (m)	INLET LENGTH (m)	TOTAL LENGTH (m)	INLET CAPACITY (%)
1.9	2.5	60	0.91	2.75	3.66	74.6
1.9	2.5	60	1.83	1.83	3.66	74.6
1.9	2.5	60	2.14	1.52	3.66	73.3
1.9	2.5	60	2.43	1.23	3.66	72.0
1.9	2.5	60	2.75	0.91	3.66	62.5
10.6	2.5	101	0.91	6.41	7.32	73.8
10.6	2.5	101	1.83	5.49	7.32	72.8
10.6	2.5	101	2.74	4.58	7.32	71.9
10.6	2.5	101	3.69	3.63	7.32	70.8
10.6	2.5	101	4.60	2.72	7.32	70.8

Table 2.3.1 indicates that the ratio between the upstream gutter length and the actual inlet length can be varied over a wide range without affecting the inlet capacity significantly. The research team did not determine an optimum depressed gutter length based on the empirical data, but suggested that further research should be conducted regarding this aspect.

In his summary Forbes admitted that the validity of some of his assumptions could be questioned, but with the good correlation between the numerical approach and the 1:6-scale model tests taken into account he was satisfied that this was a good departure point for further research. Probably the most important result from the Durban Municipality model tests, was the first concrete evidence that part of the actual inlet length could be replaced with a depressed gutter without negatively affecting the inlet capacity of the stormwater kerb inlet.

2.4 Summary

Referring to the research projects discussed so far it became obvious that certain variables do in fact affect the inlet capacity of kerb inlets more than do others. The important findings of the different research teams are listed below and are indicative of the important variables.

The findings of the three research teams can be summarised as follows:

- The variable that primarily affects the inlet capacity of a stormwater kerb inlet is the combined inlet and depressed gutter length.

- The role that the road crossfall plays in improving the efficiency of the stormwater kerb inlet is still uncertain.

- Partial interception is an economically viable option.

- Depressed gutters enhance the inlet capacity of stormwater kerb inlets.

- Results from model studies indicated that a part of the actual kerb inlet opening can be replaced with a depressed gutter.

- The development of a sound theoretical model proved to be problematic.

However, apart from the importance of the total inlet length, the research teams concluded that further research was needed in all the other areas. Some of the areas have been addressed in recent developments (Rooseboom et al, 1983 and 1988) while certain areas, e.g. the development of a suitable comprehensive theoretical model to determine the inlet capacity of stormwater kerb inlets, eluded research teams even in the most recent studies.

CHAPTER 3: RECENT DEVELOPMENTS

3.1 Development of design tools for stormwater kerb inlet design

The major shortcoming of the above-mentioned research projects was the absence of a reliable design tool with which to determine the inlet capacity of stormwater kerb inlets.

A research team at the University of Pretoria, (Rooseboom et al, 1983) solved this problem. Using the information from the above-mentioned research projects combined with their own research, which involved mainly small scale model tests, they were able to develop design curves to determine the inlet capacity for kerb inlets functioning under depressed as well as undepressed conditions, (Annexure A).

The work done at the University of Pretoria, (Rooseboom et al, 1983) was a consolidation and verification of previous work with the aim of providing the design engineer with a tool to design stormwater kerb inlets. No theoretical model emerged from this as it became even more obvious that the development of such a model would be extremely difficult. The design curves further provided the engineer with the choice of designing for either 100% interception or 80% interception.

On the negative side the design curves were primarily based on small scale model tests, especially for flow under supercritical conditions. This left the question about the influence of scale effects unanswered. It also did not deal with the uncertainties surrounding the importance of the road crossfall as all the model tests were done on a standard crossfall of 2% and the design curves catered for this crossfall only. Nevertheless, the importance of this research project cannot be underestimated as it provided the engineer with a practical design tool based on empirical data collected over a period of three decades.

3.2 Implementation of the effective inlet length concept

3.2.1 The Boksburg type kerb inlet

The research conducted by The John Hopkins University (1956), Zwamborn (1966), Forbes (1976) and Rooseboom et al (1983) concentrated on the depressed gutter that collected the water in the street and then directed it into the kerb inlet. The latest research conducted by Rooseboom et al (1988), used a new approach. The depressed gutter was replaced by a triangular transition section, (Annexure B). The transition section is designed to collect the water over a distance, then concentrating it before directing the water into the kerb opening. Thus a long overflow section is created and the expensive inlet section is minimised.

The additional advantages offered by this design are:

- It does not interfere with the traffic flow.

- Future resurfacing of the street does not reduce its inlet capacity.

The modified kerb inlet developed for the Boksburg Municipality comprises three components, viz an upstream transition section, an inlet section and a downstream transition section, (Annexure B). The upstream transition section is 6.25m in length, its crossfall is 18% and the base is 700mm wide. The inlet section is 2.5m in length and is placed at an angle of 22° relative to the direction of the street. The downstream transition section is 2.75m in length with a crossfall of 25% and the base is 500mm wide.

As mentioned before, the degree of uncertainty in respect of the inlet capacity of stormwater kerb inlets is greatest in cases where the gradients are steep and the Froude numbers are high. For their tests the research team selected a kerb inlet along one of the steepest streets and situated near an outfall where discharges from the outlet could be measured. The street had a gradient of 2.7% and a crossfall of 2.5%. The tests were performed with streetflows ranging between 75l/s and 280l/s.

An unknown variable in determining the inlet capacity of kerb inlets up to the time of this research project was the geometry of the kerb inlet along the road. With the kerb inlet rotated into the direction of the approaching flow it was found that water not only accumulated along the upstream transition and kerb inlet sections but along approximately 2/3 of the downstream transition section. This phenomenon contributed to the effectiveness of the kerb inlet.

From their results the research team deduced the following:

- The existing design chart overestimates the flow depth in the street by about 10% for a streetflow of approximately 170l/s.

- For 100% interception and a flow of 74l/s, the existing design chart predicts about the same depth of flow as was measured by the team.

- The existing design chart overestimates the inlet length by about 8% for 80% interception.

The results obtained from the full scale tests in Boksburg correlated reasonably well with the existing design curves, based on small scale model testing, (Rooseboom et al, 1983). The differences that existed were ascribed to the different street roughness coefficients. The conclusion reached was that the variable primarily affecting the inlet capacity of a kerb inlet is not the actual inlet length, but the effective inlet length, comprising the upstream transition section, the actual kerb inlet section and the greater part of the downstream transition section. This was in line with the results of Forbes (1976). This also proved that the cost of a kerb inlet could be greatly reduced by replacing part of the expensive kerb inlet structure with an inexpensive upstream transition section. From the results published by Forbes (1976), it was deduced that the upstream transition section should be approximately twice the inlet length. In the case of the Boksburg full scale tests the transition section was more than 2.5 times the inlet length without any negative effects on the inlet capacity of the kerb inlet.

The real importance of the study conducted for the Boksburg Municipality was that it proved that cost-effective kerb inlets could be designed by replacing part of the expensive kerb inlet with an inexpensive transition section. It also identified the need for further full scale testing in order to calibrate the existing design curves.

3.2.2 The Pretoria type kerb inlet

At more or less the same time that the Boksburg type kerb inlet was developed, Mr Rodney Corin of the City Engineer's Department of Pretoria developed a kerb inlet based on the same principles. This kerb inlet also comprises three components namely, an upstream transition section, an inlet section and a downstream transition section. The upstream transition section widens over a distance to a width of 800mm, with a crossfall of approximately 28%. The downstream transition is 1m in length and narrows from 450mm to 140mm over a distance of 1m with a crossfall of approximately 28%, (Annexure C).

There are however significant differences in geometry between the Pretoria and Boksburg types:

- Only a section of the Pretoria type kerb inlet faces the approaching flow. The greater length of this kerb inlet retained its side inlet geometry.

- The upstream transition section length is linked to the street gradient. The basis for determining the lengths was the radii for the "type-2" outlets as published by Rooseboom et al (1983).

- The downstream transition section of the Pretoria type kerb inlet is only one metre in length and because of its geometry does not contribute as much to the efficiency of the kerb inlet as does the downstream transition section developed by the Boksburg Municipality.

Because of the assumptions made in designing the upstream transition section of the Pretoria type kerb inlet and the differences pointed out by Rooseboom et al (1988), the decision was taken that full scale tests on the modified kerb inlet would be the only way to resolve the uncertainties surrounding the inlet capacities of modified stormwater kerb inlets. The following chapters deal primarily with the Pretoria type kerb inlet but the author will however show that the relationships developed for designing the Pretoria type kerb inlet can be applied to modified stormwater kerb inlets in general, independent of their geometry, provided that their designs are based on the "effective inlet length" concept.

CHAPTER 4: THEORETICAL CONSIDERATIONS

Through a study of the relevant literature and renewed hydraulic analysis of the problem, it became evident that flow patterns at a kerb inlet are very complex. In this chapter the author identifies the forces that affect flow patterns at a stormwater kerb inlet in order to come to an understanding of the discharge patterns.

Consider an element of water adjacent to the kerb, of unit width and length, this being the first element to be subject to unbalanced forces as it passes from the kerb side to the opening. Figure 4.1 gives more detail.

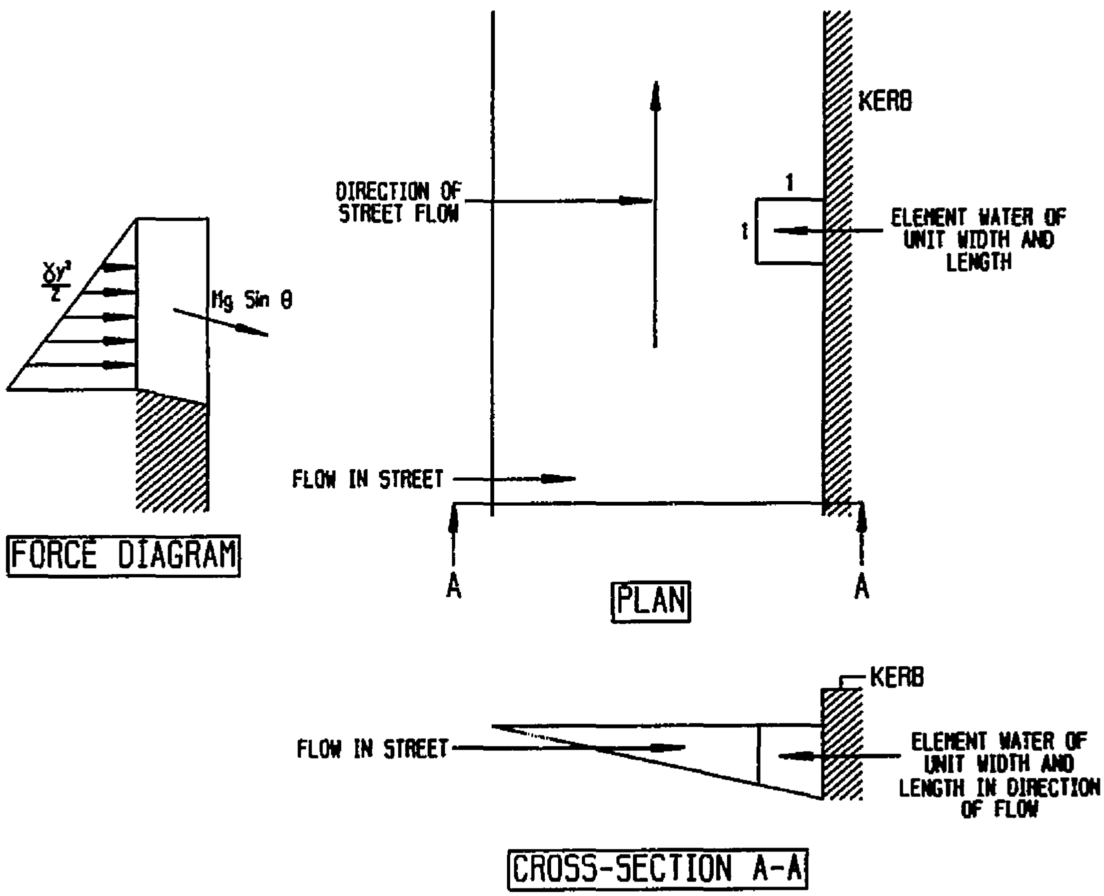


FIGURE 4.1: PLAN AND CROSS-SECTION INDICATING THE FIRST ELEMENT OF WATER SUBJECT TO UNBALANCED FORCES

Where the kerb indicated in Figure 4.1 is interrupted, the element of water is subject to unbalanced forces. In terms of Newton's second law, the rate of change in momentum is equal to the summation of the unbalanced forces:

$$\Sigma F = m \, dv/dt \qquad (4.1)$$

m : mass of element of water

dv/dt : acceleration of element

Figure 4.2 explains the choice of axes at the point where the kerb line is interrupted.

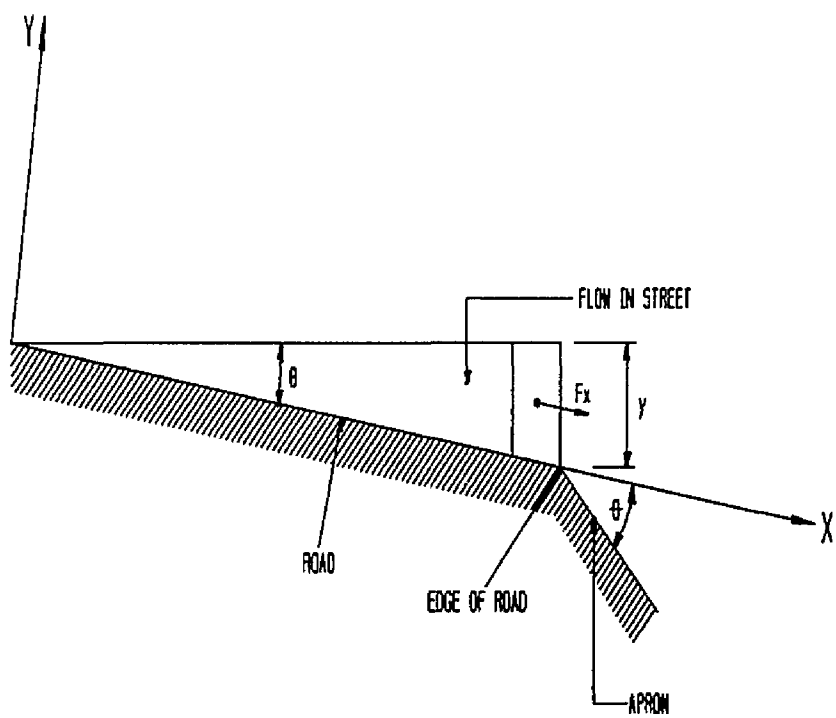


FIGURE 4.2: CROSS-SECTION WHERE KERB LINE IS INTERRUPTED

Upstream of the opening the force acting upon the element would be the weight component in the x-direction. This force is balanced by the reaction force exerted by the kerb before the kerb is interrupted:

$$F_{1X} = m.g.\sin\Theta \qquad (4.2.1)$$

Θ : angle between horizontal and x-axis
g : gravitational acceleration
F_{1X} : force due to the weight component

Due to the interruption in the kerb, a pressure gradient develops across the width of the element. This force in the x-direction for free overfall conditions equals:

$$F_{2X} = \gamma.y^2/2 \qquad (4.2.2)$$

γ : specific weight of water
y : depth of flow at kerb
F_{2X} : unbalanced force due to the pressure difference across the width of the element

It was found that the capacity did not increase by increasing the crossfall of the depressed gutter/transition section beyond 20% (Zwamborn, 1966). In effect the finding of Zwamborn (1966) indicates that the situation of a free overfall is approached. In the case of the Pretoria type kerb inlet the assumption of free overfall conditions is valid because the crossfalls of the transition and the apron along the inlet are both approximately 28%.

The sum of the unbalanced forces as a result of the pressure gradient across the width of the water element directs the flow towards the kerb inlet.

Equation 4.1 can be rewritten as:

$$\gamma . y^2/2 + m.g.\sin\Theta = m.dv_x/dt \qquad (4.3)$$

for small values of Θ, the crossfall and:

dv_x/dt : acceleration of element in the x-direction

ρ : density of water

In Equation 4.3, the effect of friction losses can be neglected as movement takes place over a very short distance. The expressions representing the unbalanced forces can be presented in terms of the depth of flow at the kerb.

$$\gamma . y^2/2 + m.g.\sin\Theta = m.dv_x/dt$$

where $m = 1.1.y.$ for an element of unit width and length

therefore $\rho.g.y^2/2 + \rho.g.y.\sin\Theta = \rho.y.dv_x/dt$

or

$$g.y^2/2 + g.y.\sin\Theta = y.dv_x/dt \qquad (4.4)$$

Equation 4.4 is applicable for an element with unit width and length.

Graphical representation of the unbalanced forces enables one to make deductions concerning the relative contribution of each of the different force components to the magnitude of the total unbalanced force (Figure 4.3).

mg Sin θ (2%) : FORCE DUE TO WEIGHT COMPONENT (CROSSFALL: 2%)
mg Sin θ (3%) : FORCE DUE TO WEIGHT COMPONENT (CROSSFALL: 3%)
$0,5\gamma y^2$: FORCE DUE TO PRESSURE GRADIENT ACROSS ELEMENT WIDTH
Sum (2%) : TOTAL UNBALANCED FORCE (CROSSFALL: 2%)
Sum (3%) : TOTAL UNBALANCED FORCE (CROSSFALL: 3%)

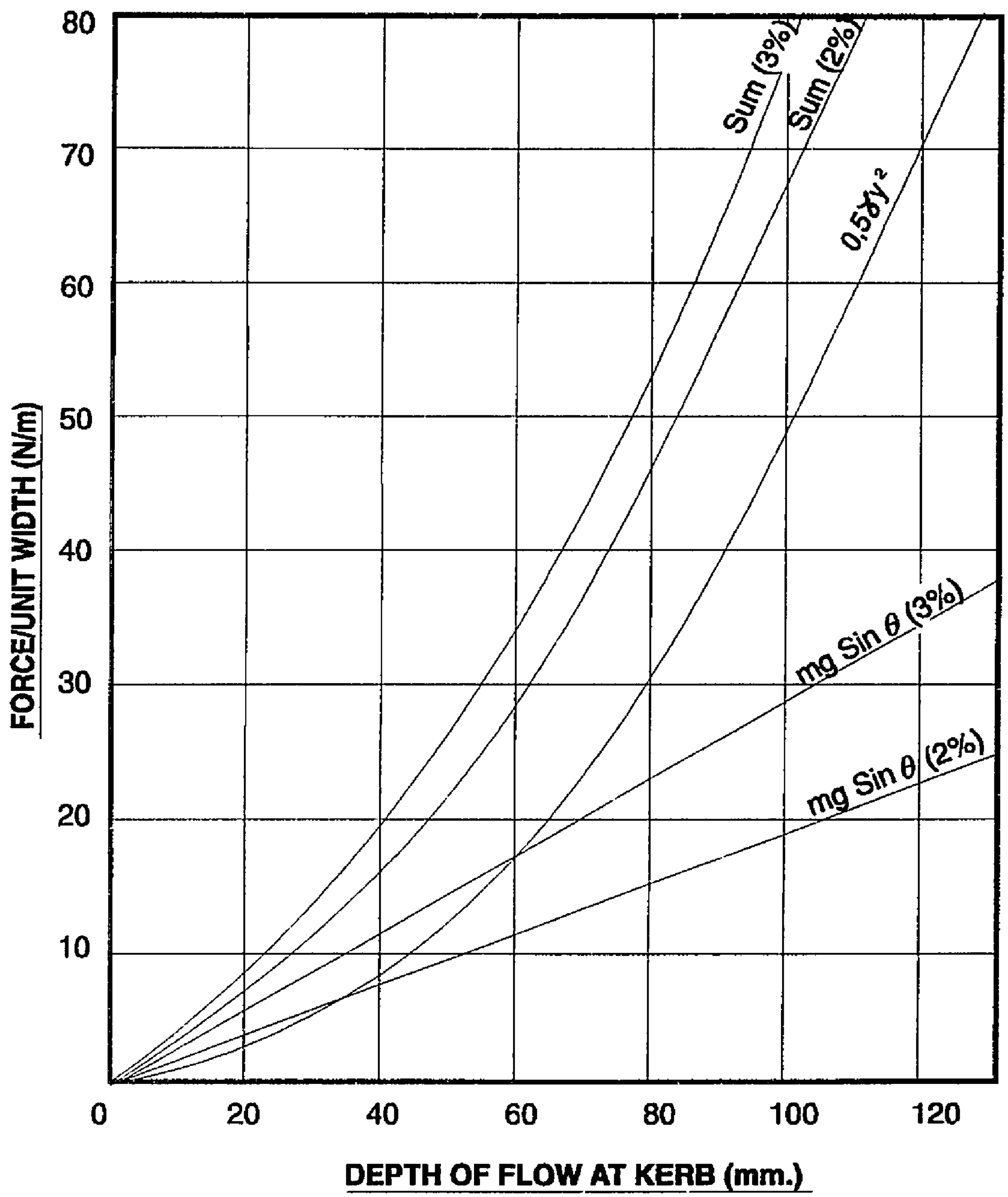


FIGURE 4.3: RELATIVE CONTRIBUTION OF FORCES ON THE TOTAL UNBALANCED FORCE CAUSING A CHANGE IN FLOW DIRECTION

The deductions are:

- At low flows with corresponding small flow depths the force representing the weight component dominates, whereas at high flows with corresponding deep flow depths the force resulting from the pressure gradient across the width of the element of water dominates.

- An increase in road crossfall and the resulting increase in the unbalanced force caused by the weight component of the element of water is directly proportional.

- A relatively large increase of, say 50%, in the road crossfall from 2% to 3% does not lead to an increase of similar order in the sum of the unbalanced forces, especially at larger flow depths (Table 4.1). This probably explains the finding of Zwamborn (1966), that the road crossfall does not affect the inlet capacity of a stormwater kerb inlet.

TABLE 4.1: THE EFFECT OF INCREASED ROAD CROSSFALL ON THE SUM OF THE UNBALANCED FORCES

FLOW DEPTH AT KERB (mm)	Σ(UNBALANCED FORCES AT 2% CROSSFALL) (N/m)	Σ(UNBALANCED FORCES AT 3% CROSSFALL) (N/m)	PERCENTAGE INCREASE IN Σ(UNBALANCED FORCES FROM 2% TO 3%) (%)
10	2.45	3.43	40.0
20	5.88	7.84	33.3
30	10.3	13.2	28.2
40	15.7	19.6	24.8
60	29.4	35.3	20.1
80	47.0	54.9	16.8
100	68.6	78.4	14.3

CHAPTER 5: FULL SCALE TESTS

The experimental data available from previous research is based primarily on small scale models. Only a minority of the tests were done on full scale models and most of the time on relatively flat gradients. The possibility of scale effects was identified as far back as 1966, by Zwamborn (1966).

For this particular study it was decided to do full scale tests on a fairly steep gradient to calibrate the existing design curves for supercritical flow conditions. All the tests were carried out on the Pretoria type kerb inlet as this kerb inlet is a good example of a modified kerb inlet utilising the "effective inlet length" concept.

5.1 Description of test facilities and procedures

The test site was situated on the premises of the Daspoort sewage disposal works in Pretoria (Annexure D). The site was ideally suited for this type of research. A road with a relatively steep gradient was available, together with adequate water supply and the layout of the road was such that the construction of flow measuring stations and the recording of flows bypassing the test kerb inlet would be possible. With the road being on City Council property, the possibility of interference by vehicular traffic was eliminated and the area was also easily accessible. During 1988 the facilities were constructed so as to facilitate proper and accurate testing. These facilities consisted of:

5.1.1 A sand pit, 10 metres in length, to facilitate the easy installation and removal of the various stormwater kerb inlets to be tested.

5.1.2 A second supplementary kerb inlet, about 60m downstream of the first kerb inlet to intercept the water bypassing the test kerb inlet. A berm was constructed across the width of the road adjacent to the second kerb inlet in order to direct as much water as possible to this kerb inlet.

5.1.3 Two approach channels, 1.50m and 0.73m wide respectively, to channel the intercepted water and the bypass water to the flow gauging stations. Both channels were constructed in compliance with the British Standards Institution, Methods of measurement of liquid flow in open channels, BS 3680: Part 4A: 1981.

5.1.4 Two rectangular thin-plate, full-width measuring weirs, precision manufactured by the CSIR according to the above specification.

5.1.5 Two point gauges placed at a distance of 4 to 5 times the head upstream from the sharp-crested weirs, operated in accordance with the above specification.

5.1.6 Water was supplied by means of a connection from the 600mm diameter water main supplying the Pretoria West station with recycled cooling water. The maximum discharge available to the test site was estimated to be in excess of 300l/s. The discharge was initially controlled by a sluice gate valve until it failed and was replaced by a butterfly valve.

Back-up service was supplied by the City Engineer's Department of the City Council of Pretoria. This consisted mainly of installing various combinations of kerb inlets between tests as well as carrying out routine maintenance on the facilities.

When the experimental work commenced, a detailed survey was conducted of the road adjacent to the test kerb inlet. The road gradient was found to be 4% and the road crossfall 2.9%. The first round of exploratory tests was conducted with the existing road gradient and crossfall. After some careful evaluation of the results it was decided to resurface the road in order to alter the crossfall to 2%. After the road crossfall was altered a full range of tests was carried out, testing various combinations of inlet and upstream transition sections. The length of the inlet sections was varied from 1 metre to 3 metres and the upstream transition length was varied from 3 metres to 6 metres. No variation in the length of the downstream transition section was tested because of its lesser contribution to the inlet capacity, as will be shown later.

All the tests were performed in a similar fashion:

- The point gauges were mounted to the measuring stations at the downstream end of the approach channels and the gauge zero readings were recorded.

- The valve was opened to introduce the maximum permissible flow into the road. This was limited either by road capacity or in the case of kerb inlets with short total lengths, by the total inlet capacity of the two kerb inlets.

- Time was allowed for the flow to stabilise, after which all values were recorded in the shortest possible time.

- The flow was then reduced and the process repeated until a discharge was reached at which 100% interception was attained at the test kerb inlet.

The recorded values consisted of:

- The gauge zero readings at the point gauges.

- The stabilised water level readings at the point gauges.

- Flow widths at different points along the road upstream from the test kerb inlet.

For the purpose of calculating the flow in each approach channel and therefore the inlet capacities of the test kerb inlet and the second kerb inlet, the SIA-formula as in BS 3680: Part 4A, (1981) for the basic weir form was used:

$$Q = (2/3).C.(2.g)^{0.5}.b.h^{1.5} \qquad (5.1.1)$$

with C : $[0.615 + 0.000615/(h + 0.0016)]\,[1 + 0.5(h/h + p)^2]$

and Q : volume rate of flow (m^3/s)

C : coefficient of discharge

g : gravitational acceleration (m/s^2)

b : measured width of the weir or approach channel in this case (m)

h : approaching head at point gauge

p : height of the weir crest relative to the canal floor (m)

The flow widths were used to calculate flow rates in the road and these calculated flow rates were compared with the flow rates determined at the thin-plate, full width measuring weirs. This was done during the initial stages of testing but because a close agreement between the measured and the calculated flows was obtained, this practise was discontinued.

5.2 Inlet capacity tests

Due to the exploratory nature of the first round of tests they will be portrayed as tests which are introductory to this research report. The emphasis will be rather on the second round of tests, which had been carried out under more stringently controlled conditions.

In the first series of tests four different combinations of upstream transition section length versus inlet section length were tested. The combinations with the respective streetflows at which 80% of the flow was intercepted are indicated in Table 5.2.1.

TABLE 5.2.1: EXPLORATORY TEST RESULTS WITH A ROAD CROSSFALL OF 2.9%

COMBINATION (No)	UPSTREAM TRANSITION LENGTH (m)	INLET SECTION (m)	DOWNSTREAM TRANSITION SECTION (m)	TOTAL LENGTH (m)	STREETFLOW AT WHICH 80% INTERCEPTION IS ATTAINED (l/s)
1	4	2	1	7	65.0
2	4	3	1	8	85.0
3	6	2	1	9	127.5
4	6	3	1	10	165.0

In the second series of tests, 12 different combinations were tested and the various combinations with the respective streetflows at which 80% of the flow was intercepted are portrayed in Table 5.2.2

TABLE 5.2.2: TEST RESULTS WITH A ROAD CROSSFALL OF 2.0%

COMBINATION (m)	UPSTREAM TRANSITION LENGTH (m)	INLET LENGTH (m)	DOWNSTREAM TRANSITION LENGTH (m)	TOTAL LENGTH (m)	STREETFLOW AT WHICH 80% INTERCEPTION IS ATTAINED (l/s)
1	3	1	1	5	14.3
2	3	2	1	6	30.6
3	3	3	1	7	37.4
4	4	1	1	6	43.5
5	4	2	1	7	60.3
6	4	3	1	8	76.7
7	5	1	1	7	70.1
8	5	2	1	8	87.8
9	5	3	1	9	108.0
10	6	1	1	8	87.3
11	6	2	1	9	108.7
12	6	3	1	10	129.2

Both Tables 5.2.1 and 5.2.2 are summaries of the test results. The complete record of all test results is detailed in Annexure F.

5.3 Evaluation of results

From observations made during the tests it was deduced that approximately half (1/2) of the downstream transition section contributes to the inlet capacity in the case of the Pretoria type kerb inlet, (Annexure C). This has a major effect on the test results as it means that the "effective inlet length" comprising the upstream transition section, the inlet section and half of the downstream transition section is 0.5m shorter than the total length indicated by Tables 5.2.1 and 5.2.2. The inlet capacities from the full scale model tests were compared with the inlet capacities predicted by the existing design curves for undepressed kerb inlets as published in the Road Drainage Manual of The South African Road Board.

This last-mentioned comparison is given in Table 5.3.1. All inlet capacities are the streetflows at which 80% of the flow is intercepted and in each case the contribution of the downstream transition section is assumed to be 0.5m.

TABLE 5.3.1: COMPARISON OF INLET CAPACITIES PREDICTED BY THE EXISTING DESIGN CURVES AND INLET CAPACITIES OBTAINED FROM FULL SCALE MODEL TESTS

EFFECTIVE INLET LENGTH (m)	INLET SECTION (m)	UPSTREAM TRANSITION SECTION (m)	EXISTING DESIGN CURVES (l/s)	FULL SCALE MODEL TESTS, 2.0% ROAD CROSSFALL (l/s)	FULL SCALE MODEL TESTS, 2.9% ROAD CROSSFALL (l/s)
4.5	1	3	18.0	14.3	
5.5	2	3	29.0	30.6	
	1	4		43.5	
6.5	3	3	42.0	37.4	
	2	4		60.3	65.0
	1	5		70.1	
7.5	3	4	57.0	76.7	85.0
	2	5		87.8	
	1	6		87.3	
8.5	3	5	74.0	108.0	
	2	6		108.7	127.5
9.5	3	6	93.0	129.2	165.0

Analysing Table 5.3.1 the following can be concluded:

5.3.1 In general the inlet capacities from the full scale model tests are higher than the inlet capacities predicted by the existing design curves, except in the case where a 3m upstream transition section was used.

5.3.2 The difference in inlet capacity for constant effective inlet lengths but with different upstream transition lengths namely 4, 5 and 6 metres was negligible.

5.3.3 The increase in road crossfall from 2% to 2.9% affected the inlet capacity of the Pretoria type kerb inlet significantly only in the case of the longer effective inlet lengths namely, 8.5m and 9.5m.

A matter of concern was the test results obtained for kerb inlets with a 3m upstream transition section. As indicated in Table 5.3.1 the increase in inlet capacities predicted by this combination was not in line with those for the other combinations. As this was the last combination to be tested the model was still in place and could be investigated. The investigation showed a defect in the construction of the upstream transition section directly next to the road edge. The first 0.5m of the transition section was 20mm higher instead of 20mm lower than the road edge and this affected the inlet capacity of all the kerb inlets constructed with a 3m upstream transition section in the test programme. This may seem to be a small construction defect but in fact does not conform to the allowable tolerances as described in the Standard Specifications for Municipal Engineering Works (1991). This problem was not encountered before and can only be ascribed to the very limited time available to the construction team building the test kerb inlets. The inlet capacities predicted for constant effective inlet lengths with 4, 5 and 6m upstream transition sections indicated that the same construction problem did not occur in the other model tests. This defect was only discovered at the end of the experimental programme and because of the limited funds available, this particular combination could not be reconstructed. It was therefore decided to proceed with the analysis of the data, taking this defect into

5.4 Processing of results

In general, the full scale model tests indicated that the existing design curves are conservative, i.e. the inlet capacities as observed during the tests were higher than those predicted by the existing design curves. It must be stated that the most conservative envelope of small scale model test results was used in drawing up the existing design curves. It has also been shown that capacities tend to be underestimated in small models. In Table 5.4.1 the difference in inlet capacity for each combination is indicated.

TABLE 5.4.1: RELATIVE DIFFERENCES BETWEEN PREDICTED INLET CAPACITIES OBTAINED FROM EXISTING DESIGN GRAPHS AND INLET CAPACITIES OBTAINED FROM FULL SCALE MODEL TESTS

EFFECTIVE INLET LENGTH (m)	INLET SECTION (m)	UPSTREAM TRANSITION SECTION (m)	S A R B UNDEPRESSED KERB INLET (l/s)	C C P-KERB INLET, ROAD CROSSFALL 2% (l/s)	DIFFERENCE IN INLET CAPACITY AT 80% INTERCEPTION (%)
4.5	1	3	18.0	14.3	-21
5.5	2	3	29.0	30.6	6
	1	4		43.5	50
6.5	3	3	42.0	37.4	-11
	2	4		60.3	44
	1	5		70.1	67
7.5	3	4	57.0	76.7	35
	2	5		87.8	54
	1	6		87.3	53
8.5	3	5	74.0	108.0	46
	2	6		108.7	47
9.5	3	6	93.0	129.2	39

The determination of a factor to adapt the existing design curves needs to provide for the shortcomings of the full scale model, which were found to be:

5.4.1 All the tests were conducted with a street gradient of 4%. Although this gradient is steep enough to ensure that supercritical flow exists in virtually all recorded conditions, the danger of being too optimistic in determining a factor to be applied to existing design curves, must be kept in mind.

5.4.2 Although the approach channels to the measuring weirs were constructed to meet the specifications, wave action was observed periodically in the approach channel conveying the flow intercepted by the test kerb inlet. In spite of the fact that care was taken not to take any water surface elevation readings during such wave action, it is impossible to assume that the occurrence of small wave action can be ruled out. When adjusting the existing design curves this must be borne in mind.

5.4.3 The road upstream of the two kerb inlets as well as the two approach channels were cleaned on a regular basis but some silt was usually found in the approach channels directly upstream of the two thin-plate rectangular weirs after a day of testing. The extent of the siltation was however limited and the effect of it on the results should be minimal.

5.4.4 The most correct way of accommodating the construction defect mentioned previously in this chapter was to exclude those results from the experimental data to be used to calibrate the existing design curves. This negatively affected the representativeness of the experimental data.

5.4.5 In adjusting design curves of such importance, the possibility of human error in the experimental process must never be overlooked and any adjustment to the existing design curves must be based on a realistic approach.

Care was taken throughout the test programme to minimise the effects of the above-mentioned shortcomings. However, as mentioned before a construction defect was detected after completion of the test programme. Because of the extreme care taken during the test programme, the effects of these shortcomings on the accuracy of the test results, other than the construction defect can be regarded as minimal. The effect of the construction defect on the adjustment of the existing design curves will be addressed in Chapter 6.

CHAPTER 6: REVISION OF EXISTING DESIGN TOOLS

The full scale tests described in Chapter 5 show that there is a general trend for the existing design curves to be conservative. Except for the three tests where a construction defect influenced the results, the increase in inlet capacity was in the range 35 to 67 percent. The mean increase for the sample excluding the results influenced by the construction defect was 48% and the 95% confidence interval for the mean was from 41 to 56 percent (Annexure J). It therefore seems logical to make an upward adjustment to the inlet capacity of the existing design curves by say 40%.

The sensitivity of the inlet capacity to small construction defects was however illustrated by the full scale tests. In these tests an error of 40mm in the height of the first 0.5m of a 3m upstream transition section of the kerb inlet, caused an average reduction of 9% in the inlet capacity relative to the existing design curves. This is in contrast with an average increase of 48% in the remaining tests. This sensitivity to the accuracy of the construction has lead the author to make the following practical recommendations:

- To increase the inlet capacity of the existing design curves by 30% instead of the 40% indicated above.

- To take extreme care during construction to ensure that the upstream transition sections are never built higher than specified.

Although the basis for the recommendation of a 30% increase is not very scientific, it is in the opinion of the author a realistic recommendation. As will be shown later it also is in line with the findings of Rooseboom et al (1988).

An adjustment to the existing design curves of 30% for those effective inlet lengths tested in this research project, resulted in the following inlet capacities for a 4% road gradient and a 2% road crossfall. (Table 6.1)

TABLE 6.1: INLET CAPACITIES PREDICTED BY EXISTING DESIGN CURVES AFTER BEING ADJUSTED UPWARDS BY 30%

EFFECTIVE INLET LENGTH IN METRES	STREETFLOW AT 80% INTERCEPTION IN l/s FROM EXISTING DESIGN CURVES	STREETFLOW AT 80% INTERCEPTION IN l/s AFTER A 30% UPWARD ADJUSTMENT OF EXISTING DESIGN CURVES
4.5	18.0	23.4
5.5	29.0	37.7
6.5	42.0	54.6
7.5	57.0	74.1
8.5	74.0	96.2
9.5	93.0	120.9

The existing design curves comprise two sets of graphs and the graphs dealing with the inlet lengths cannot be adjusted independently of the graphs dealing with the streetflows at various road gradients. The graphs dealing with streetflows originated from applying the Manning equation to a single cross-section which is strictly speaking not correct. For the purpose of modifying this graph the Chezy equation was used and applied to a cross-section divided into eight sub cross-sections. The absolute roughness coefficients were chosen from Figure 3.8; Average roughness coefficients for rough turbulent flow: Rooseboom et al (1983). The values were:

$$q = 18 \, [\log (12R)/k] \, (R.S)^{0.5} \qquad (6.1.1)$$

with q : flow per sub cross-section

R : hydraulic radius

k : absolute roughness parameter

S : gradient ($=S_f$: energy slope) for uniform flow

and $k_{ASPHALT}$: 0.003m

$k_{CONCRETE}$: 0.001m

Using these absolute roughness coefficients the calculated streetflows compared well with the recorded streetflows obtained at the test site.

With the above preparatory work completed the actual modifications to existing design curves could proceed. This included the manual adjustment of the inlet capacity graph for each of the various inlet lengths. The eventual result however was satisfactory and is attached to this document in Annexure G.

An interesting exercise was the determination of the relationship between the difference in inlet capacity and the required inlet length of the existing and revised design curves. This was done by comparing the inlet lengths at the adjusted kerb inlet capacities with the required inlet lengths obtained from the existing design curves. This relationship is indicated in Table 6.2.

TABLE 6.2: REQUIRED EFFECTIVE INLET LENGTHS OBTAINED FROM FULL SCALE MODEL TESTS AND FROM EXISTING DESIGN CURVES

ADJUSTED STREETFLOW AT 80% INTERCEPTION (l/s)	RECORDED EFFECTIVE INLET LENGTH (m)	REQUIRED INLET LENGTH (EXISTING DESIGN CURVES) (m)	DECREASE IN REQUIRED INLET LENGTH (%)
23.0	4.5	5.0	11.1
38.0	5.5	6.1	10.9
55.0	6.5	7.2	10.8
74.0	7.5	8.4	12.0
96.0	8.5	9.5	11.8
121.0	9.5	10.6	11.6

From Table 6.2 it can be deduced that an increase of 30% in inlet capacity has the same result as an approximate decrease of 11% in the required effective inlet length. This agrees with the findings of Rooseboom et al (1988), where they predicted a decrease of about 8% in required effective inlet length. Re-analysis of their data proved that the decrease in required effective inlet length is closer to 11.5%. This is an important finding since it confirms that at two different test sites where the "effective inlet length" concept was tested, essentially the same results were obtained. The only difference between the two research projects are the reasons given for the difference in predicted inlet capacity between the full scale model tests and the existing design curves of the kerb inlets. In the case of the Boksburg tests the difference was ascribed to different street roughness coefficients being applied, whereas in the case of the Pretoria tests the opinion is that scale effects are primarily responsible for the difference in inlet capacities. This is supported by the good correlation between the calculated flows in the road obtained from the Chezy equation, with the selected absolute roughness coefficient for asphalt surfaces approximately the same as the average value for the roughness coefficients for asphalt surfaces as given by Rooseboom et al (1983), and the recorded flows at the measuring site.

The remaining area that requires attention at this stage is the increase in road crossfall and its effect on the inlet capacity of stormwater kerb inlets.

Referring back to The John Hopkins University (1956), Zwamborn (1966), the basic theoretical model developed in Chapter 4 regarding the increase in the road crossfall and its resultant effect on the unbalanced forces affecting the behaviour of an element of water and the results from the full scale model tests indicated in Chapter 5, the following comments concerning the effect of road crossfall on the inlet capacity of kerb inlets can be made:

- Full scale model tests indicated a significant effect on the inlet capacity of the kerb inlets by the crossfall only in the case of the very long effective inlet lengths, i.e. 8.5m and 9.5m.

- The theoretical model indicated that an increase in road crossfall would have a limited influence on the inlet capacity of kerb inlets.

- Zwamborn (1966) stated in his research that the road crossfall does not affect the inlet capacity of stormwater kerb inlets.

- The John Hopkins University (1956) assumed in their theoretical model that the road crossfall was the most important variable affecting the inlet capacity of stormwater kerb inlets, but the empirical constant in their theoretical model, based on the road crossfall alone, was not sensitive to a change in the road crossfall.

From the above it became obvious that if the road crossfall affects the inlet capacity of stormwater kerb inlets the effect is not very marked. This can be substantiated by the following argument. With a road gradient of 4% and an increase from 2% to about 3% only the longer effective inlet lengths gave a significant increase in inlet capacity. With a further increase in road gradient the Froude number of the flow would increase, making it more difficult for the water to change direction. With the practical limitations placed on the total length of a stormwater kerb inlet in urban areas it would mean that in the case of very steep roads it would become virtually impossible to make a stormwater kerb inlet long enough so as to allow the effect of the increase in road crossfall to come into play.

It is therefore the firm belief of the author that an increase in the road crossfall does not provide the engineer with a tool to improve the design of stormwater kerb inlets to such an extent that a significant financial saving can be accomplished.

After the increases in the inlet capacity of stormwater kerb inlets were quantified and the existing design curves were adjusted it left the author with a set of updated design curves that were still not "user friendly", especially for the engineer who does not use them on a daily basis. The development of a design curve that is "user friendly" and has practical advantages, left the author with a challenging task.

CHAPTER 7: DEVELOPMENT OF NEW DESIGN TOOLS

7.1 Development of new design curves

As mentioned previously, the design engineer when using the existing design curves was faced with certain practical problems. These were mainly:

- The determination of the required inlet capacity or, alternatively, the required inlet length, requiring the use of two sets of graphs.

- The conservative answers provided for inlet length and inlet capacity under supercritical flow conditions.

The objectives in developing the new design curves were:

- To depict all the relevant variables, namely inlet length, streetflow, street gradient, Froude number and flow depth on a single set of design curves.

- The use of a linear scale to make the curves more "user friendly".

- The development of design curves applicable to an urban environment.

- To highlight the importance of supercritical flow in the design of stormwater kerb inlets.

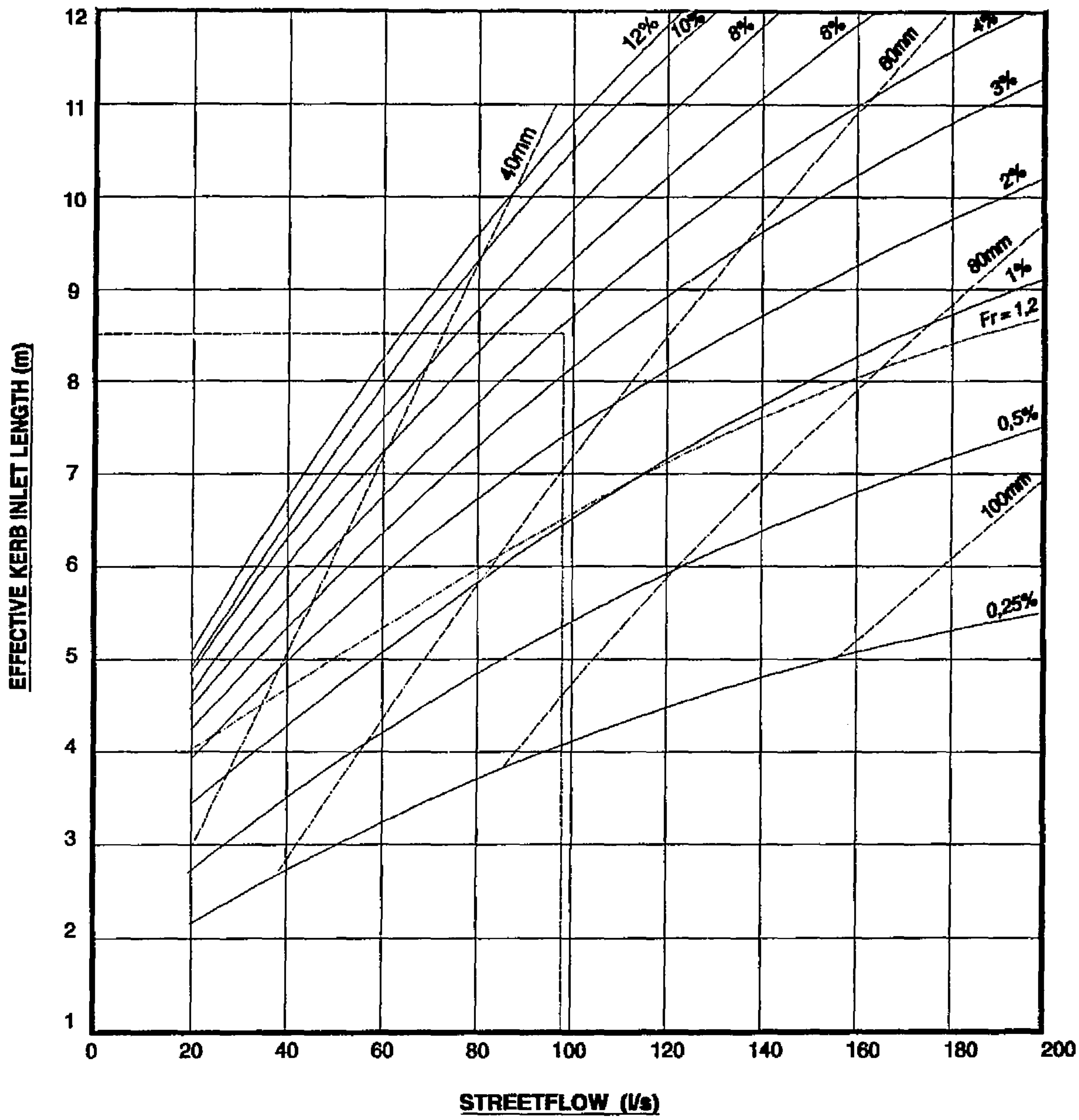
In developing the new design curves the modified existing design curves were the primary source of available data.

Following some careful consideration and discussions with various engineers engaged in the field of stormwater drainage it was decided to present the design curves as a two-dimensional graph with the independent variable, namely streetflow, on the horizontal axis and the dependent variable, namely the effective inlet length, on the vertical axis. The road gradient, the kerb flow depth and the Froude number are represented in the axial plane which is defined by the two axes. The data for streetflows up to 200l/s, for effective inlet lengths from approximately 2.5 metres to 12 metres and for road gradients varying from 0.25% to 12% was extracted from the modified existing curves and plotted as the new design curves. After this was done, the kerb flow depths were plotted on the new design curves utilising the Chezy equation and the Froude numbers using the version of the equation for a section of unit width:

$$Fr = v/(g.y)^{0.5} \qquad (7.1.1)$$

with
- v : average velocity of streetflow in m/s.
- g : gravitational acceleration in m/s^2.
- y : flow depth against the kerb in m.

Figure 7.1.1 shows the format of the new set of design curves with a complete full scale version attached as Annexure H. An example explaining the use of the new design curves is given at the end of this chapter.



———————— EFFECTIVE KERB INLET LENGTH AT SPECIFIED ROAD GRADIENT

— — — DEPTH OF FLOW AT KERB

—— — —— FROUDE NUMBER = 1.2

CURVES DEPICT 80% INTERCEPTION AT SPECIFIED STREETFLOW

THE USE OF THE GRAPH IS EXPLAINED IN EXAMPLE 7.2.1

FIGURE 7.1.1: NEW DESIGN CURVES

Referring to Annexure H the following has become apparent:

- The "user-friendly" appearance, i.e. the use of a linear scale and the fact that all relevant variables are indicated on a single set of curves.

- The determination of the effective inlet length for a given streetflow or the inlet capacity for a specified effective kerb inlet length is a straight forward process with little possibility of error.

The development of the new design curves not only solved a few practical problems but also served the purpose of clarifying certain uncertainties and areas where a lack of knowledge was apparent in the design of stormwater kerb inlets. These are mainly:

7.1.1 In practice the vast majority of South African kerb inlets function under supercritical flow conditions. Annexure G indicates that for a discharge of only 55 l/s on a road with a gradient of 1%, supercritical flow conditions do in fact exist.

7.1.2 The inlet capacity per unit length of a kerb inlet increases with increasing inlet length (Table 7.1.1). This indicates that the efficiency of a kerb inlet increases with length and this confirms that an optimum kerb inlet length cannot be determined from a hydraulic point of view but that the determination of an optimum inlet length should be dictated by financial and practical norms.

TABLE 7.1.1: INCREASING EFFICIENCY WITH AN INCREASE IN EFFECTIVE INLET LENGTH

ROAD GRADIENT (%)	EFFECTIVE KERB INLET LENGTH (m)	STREETFLOW AT WHICH 80% OF FLOW IS INTERCEPTED (l/s)	INLET CAPACITY PER UNIT LENGTH (l/s.m)
4	5	30	6.0
4	6	47	7.8
4	7	65	9.3
4	8	86	10.8
4	9	109	12.1
4	10	135	13.5
4	11	164	14.9

7.1.3 Paragraph 7.1.2 confirms that the effective kerb inlet length remains the most important variable affecting the inlet capacity of a stormwater kerb inlet as well as the assumption in Chapter 4 that the velocity in a longitudinal direction decreases. This in turn leads to a decrease in the momentum in the longitudinal direction which consequently improves the ability of an element of water to change direction.

A possible shortcoming in the new design curves is that they do not cater for 100% interception. This is because they are based solely on the existing design curves which do not predict 100% interception for supercritical flow. The existing design curves were not developed for 100% interception because it was found that it was not economically viable to intercept all the water at each kerb inlet. It is however generally accepted policy that all the water needs to be intercepted at a street intersection. Therefore a way of determining the 100% interception had to be found. To do this the street flow at which 80% of the flow is intercepted were determined using the new design curves. Factors of 1.1 and 1.12, namely a 10% and 12% increase respectively in effective inlet length, were applied to the selected inlet lengths and the streetflow at which 80% of the flow was intercepted and the actual flow intercepted was then determined. In the case of a 10% increase in the effective inlet length the predicted actual inlet capacity was in most of the cases lower than the original streetflow at which 80% of the flow was intercepted. In the case of a 12% increase in the effective inlet length, the actual inlet capacities determined were virtually always higher than the original streetflow at which 80% of the flow was intercepted. Annexure I shows the increase in actual inlet capacity for both a 10% and a 12% increase in effective inlet length. Table 7.1.2 shows the data for an increase of 12% in effective inlet length only.

TABLE 7.1.2: INCREASED INLET LENGTHS TO FACILITATE 100% INTERCEPTION

EFFECTIVE INLET LENGTH (m)	ACTUAL INLET CAPACITIES WITH AN INCREASE OF 12% IN EFFECTIVE INLET LENGTH (FLOWS DISPLAYED IN BRACKETS INDICATE STREET FLOWS AT WHICH 80% OF FLOW IS INTERCEPTED) (l/s)						
	ROAD GRADIENT (%)						
	0.5	2.0	4.0	6.0	8.0	10.0	12.0
3.0 3.4	(28) 31	- -	- -	- -	- -	- -	- -
4.0 4.5	(55) 56	(22) 26	- -	- -	- -	- -	- -
5.0 5.6	(87) 87	(42) 44	(30) 32	(25) 26	(22) 24	(21) 22	(20) 22
6.0 6.7	(125) 126	(64) 66	(47) 47	(39) 41	(36) 36	(33) 34	(32) 32
7.0 7.8	(173) 176	(90) 90	(65) 65	(56) 56	(50) 50	(46) 46	(43) 43
8.0 9.0	- -	(118) 120	(86) 87	(74) 74	(65) 66	(60) 61	(57) 57
9.0 10.1	- -	(150) 152	(109) 110	(93) 93	(82) 82	(75) 75	(71) 71
10.0 11.2	- -	- -	(135) 136	(114) 114	(100) 99	(92) 91	(85) 86
11.0 12.3	- -	- -	- -	(137) 139	(120) 120	(110) 110	(101) 102

From Table 7.1.2 it can be seen that an increase of 12% in the effective inlet length leads to an increase in inlet capacity sufficient to allow 100% interception of the original streetflow in virtually all the cases.

It should however be noted that it is not recommended to design for 100% interception under normal conditions as this will lead to an expensive system. Stormwater kerb inlets functioning under normal conditions should still be designed for 80% interception in the case of supercritical flow and only in the case of kerb inlets placed in critical positions should the effective inlet length be increased by 12% to ensure that no water bypasses the stormwater kerb inlet. As stormwater kerb inlets in streets with supercritical flow, will not be designed for 100% interception on a regular basis, the author considers it unnecessary to develop curves for 100% interception. If 100% interception is required the engineer should determine the effective inlet length required to intercept 80% of the design flow, and then add 12% to the length determined for 80% interception to provide for 100% interception.

The new design curves (Annexure H) and the modified existing design curves (Annexure G) give the same answer during design. The need for a new set of design curves was identified by engineers designing stormwater kerb inlets in an urban environment. It was shown earlier in this chapter that most kerb inlets function under supercritical flow conditions, where the existing design curves proved to be inadequate. The advantage of the new design curves are that they were developed specifically for design under supercritical flow conditions and thus resulting in an improved design process.

With the modification of the existing design curves and the development of a new set of design curves, the drawing up of guidelines for the design of upstream transition section lengths remained the only outstanding item in this research project. This matter is addressed in the final section of this chapter.

7.2 Guidelines for the design of upstream transition lengths

As was mentioned in Chapter 4, a description of the flow characteristics at a stormwater kerb inlet is virtually impossible without resorting to a large number of simplifying assumptions because of the complex nature of the flow patterns. Drawing up guidelines for the design of upstream transition sections based purely on a theoretical approach presents a similar problem. It was therefore decided to base the new guidelines on experimental results obtained in this research project and from data by Forbes (1976) and Rooseboom (1988).

When guidelines of this nature need to be drawn up it is important to realise that the geometry of the upstream transition section plays an important role in the hydraulic characteristics of the transition section. This was illustrated by The John Hopkins University (1956) when different types of depressed gutters were investigated.

In the case of a depressed gutter section with a small cross-section it is obvious that this type of gutter section will be submerged over a shorter distance than will a transition section with an increasing cross-sectional area in a downstream direction. It is therefore not possible to draw up a uniform set of guidelines which is applicable to all types of upstream transition sections. The ideal would be to have basic guidelines that are applicable to most types of upstream transition sections and leave it to the design engineer to specify guidelines for each type of upstream transition section.

In this chapter basic guidelines will be identified with specific reference to the Pretoria type upstream transition section.

In his research done in conjunction with Durban Municipality, Forbes (1976) determined that the depressed gutter could be extended in an upstream direction from the inlet section. From his results, discussed in Chapter 2, it can be deduced that for a depressed gutter with a width of 305mm and a maximum depth of 61mm the gutter could be extended upstream to about twice the actual inlet length without adversely affecting the inlet capacity of the kerb inlet. These results were based on 1:6-scale models and included road gradients varying from 1.9% to 12.1%.

In 1988 Rooseboom et al established from a full scale model that the upstream transition section with an increasing cross-sectional area in a downstream direction can be made 2.5 times the inlet length without negatively affecting the inlet capacity of the stormwater kerb inlet. An important feature of the Boksburg type kerb inlet was the length of 6.25m of the upstream transition section. This upstream section was constructed in a street with a gradient of 2.7%. Under these circumstances the upstream transition section could not be submerged and therefore negatively affect the inlet capacity of the kerb inlet. This indicated that the transition section with an increasing cross-sectional area in a downstream direction is more effective than the constant width depressed gutter as used by Forbes (1976). The upstream transition section used in the Pretoria type kerb inlet is very similar to the transition section used in the Boksburg type kerb inlet (Table 7.2.1) and the hydraulic characteristics of these two transition sections should be virtually identical (Annexures B and C). In this research project upstream transition sections up to 6m in length were tested and the upstream transition section length was extended to 6 times the inlet length with no significant decrease in inlet capacity of the stormwater kerb inlet. This confirms that for supercritical flow the upstream transition section can safely be extended for a distance of 6 times the inlet length upstream of the stormwater kerb inlet without any negative effect on the inlet capacity of the kerb inlet. This offers significant savings in the construction of kerb inlets because it means that a very large portion of the expensive inlet section can be replaced by an inexpensive upstream transition section.

TABLE 7.2.1: COMPARISON OF IMPORTANT VARIABLES FROM THE BOKSBURG AND PRETORIA FULL SCALE MODEL TESTS

DESCRIPTION OF TEST KERB INLETS AND IMPORTANT VARIABLES	TYPE OF STORMWATER KERB INLET	
	PRETORIA TYPE KERB INLET	BOKSBURG TYPE KERB INLET
ROAD GRADIENT	4%	2.7%
ROAD CROSSFALL	2%	2.5%
INLET LENGTH	1m to 3m	2.5m
UPSTREAM TRANSITION LENGTH	3m to 6m	6.25m
UPSTREAM TRANSITION SECTION CROSSFALL	27.8%	18%
RATIO (UPSTREAM TRANSITION SECTION LENGTH / INLET SECTION LENGTH)	1 to 6	2.5
TRANSITION SECTION BASE WIDTH AT UPSTREAM END OF INLET SECTION	800mm	700mm

From Table 7.2.1 it can be seen that the model tests carried out on the Pretoria and Boksburg type kerb inlets can be used to formulate certain guidelines for a triangular-shaped upstream transition section. These test results cannot be used to determine suitable upstream transition section lengths for stormwater kerb inlets placed in streets with gradients flatter than 3% because no tests were carried out on flatter road gradients. The guidelines for determining the upstream transition section lengths will be based primarily on data from previous research work.

With the available information, flow in a normal street can be classified into three distinct categories, namely:

7.2.1 subcritical flow,

7.2.2 an intermediate category with an upper limit of 3% in road gradient and a lower limit where the flow is "sufficiently" supercritical or in other words where the Froude number of the streetflow exceeds 1.2

7.2.3 supercritical flow in roads with gradients steeper than 3%.

For the flow in the subcritical category the author has come to the conclusion that the length of the upstream transition section should be limited to twice the inlet length with an absolute maximum length of 4 metres, this being based on practical considerations and on the data from Forbes (1976). For the supercritical category the maximum theoretical length of the upstream transition section must be limited to six times the inlet length. From a practical point of view it seems at this stage that the length of an upstream transition section should be limited to 6 metres for urban applications. As the intermediate category forms a transition between the above-mentioned categories, it must be handled with great care. If the "performance" of a 6 metre upstream transition section in both the Boksburg and Pretoria tests is considered, it is recommended that the upstream transition section in the intermediate category be limited to twice the inlet length with an absolute maximum of 5 metres. The restriction imposed on the ratio of the upstream transition section to the inlet section may seem conservative but as no experimental results exist in this category a factor of 2, based on the data from previous research, seems to be a safe approach. Table 7.2.2 can be used in conjunction with the new design curves when designing stormwater kerb inlets in order to decide on the most suitable upstream transition section.

TABLE 7.2.2: GUIDELINES FOR THE USE OF UPSTREAM TRANSITION SECTION LENGTHS

PARAMETERS	CATEGORY		
	FROUDE NUMBER < 1.2	FROUDE NUMBER > 1.2 ROAD GRADIENT < 3%	ROAD GRADIENT > 3%
MAXIMUM RATIO (UPSTREAM TRANSITION SECTION LENGTH / INLET SECTION LENGTH)	2	2	6
ABSOLUTE MAXIMUM LENGTH OF TRANSITION	4m	5m	6m

Table 7.2.2 illustrates the basic guidelines for the design of triangular upstream transition sections similar in geometry to the Boksburg and Pretoria type transition sections. For transition sections with a different geometry these guidelines will probably have to be modified depending on the difference in geometry of the transition sections.

Table 7.2.2 must be used in conjunction with the new and existing design curves as shown in Annexures G and H. An example in this regard is given and applies to the design of the Pretoria type stormwater kerb inlet.

EXAMPLE 7.2.1

A stormwater kerb inlet is required along a street with a gradient of 4% and a street flow of 98l/s.

STEP 1: Determine the required effective inlet length from Annexure H:

Effective inlet length = 8.5m

STEP 2: Determine the actual length of the structure:
From Chapter 5, add 0.5m to the effective inlet length as only half of the downstream transition section contributes to the inlet capacity of the stormwater kerb inlet.

Total length of structure = 9.0m
Downstream transition section for the Pretoria type kerb inlet is 1 metre as explained in Chapter 3.

STEP 3: From Table 7.2.2 the upstream transition section length can be determined:

Upstream transition length = 6m

STEP 4: Determine the actual inlet length:

Actual inlet length = Total length of structure - length of upstream transition section - length of downstream transition section
= 9.0 - 6.0 - 1.0
= 2.0m

From Example 7.2.1 it can be seen that once the flow in the street is known from hydrological calculations it takes a mere 4 steps to design the structure that would intercept 80% of the specified streetflow.

With the information in this chapter the design engineer is in a position to effectively design a stormwater kerb inlet in a short space of time.

CHAPTER 8: CONCLUSIONS AND RECOMMENDATIONS

At the outset of this research project certain uncertainties existed in the design of stormwater kerb inlets. These were formulated in the form of four questions, which can now be answered, albeit to a limited extent in some cases. In this chapter the answers to these questions will be followed by a short exposition on the outcome of the research project.

8.1 As far as the question regarding accuracy of the existing design curves is concerned, it can be stated that these are applicable to modified stormwater kerb inlets. As these curves are based primarily on small scale model tests they are subject to scale effects and based on the full scale tests an upward adjustment of 30% in the inlet capacity determined according to these curves can be justified.

8.2 Previous research and extensive tests have led to the conclusion that a large part of the expensive inlet section can be replaced by an inexpensive equivalent upstream transition section without any negative effect on the inlet capacity of the stormwater kerb inlet. In the case of supercritical flow the tests have indicated that the upstream transition section could be extended to as much as six times the inlet length for the same overall length and thus offering a substantial saving in the cost of stormwater kerb inlet structures. An added advantage of using long transitions rather than inlets is that they can be crossed partly by vehicles e.g. in front of garages.

8.3 As the test site did not offer the option of varying the gradient of the road it was not possible to determine the precise effect of the road gradient on the allowable upstream transition length. With the aid of previous research data it was however possible to determine basic guidelines for the design of upstream transition sections. This research project served the important function of proving that the restrictions imposed on the length of upstream transition sections can be disposed of to a large extent as was indicated in Chapter 7.

8.4 The least successful part of this project was the attempt to develop comprehensive mathematical formulae to describe the inlet capacity of kerb inlets. This proved to be impossible without a substantial number of assumptions being made, thereby placing the theoretical soundness of the model in question.

Summarising, it can be stated that the existing design curves after being verified and modified to some extent, are applicable to the design of conventional as well as modified stormwater kerb inlets. Furthermore it was proved that the expensive, conventional stormwater kerb inlet can be modified by replacing a major part of the expensive inlet section with an equivalent inexpensive transition section, thereby reducing the cost of the inlet by a substantial amount without any negative effect on the inlet capacity of the stormwater kerb inlet. What is important is that this is especially the case for supercritical flow, an area in which engineers over the years had difficulty in providing cost-effective stormwater kerb inlets.

As in most research projects, further work in this field needs to be done. In the short term, selected large scale model tests or full scale tests on streets with steep gradients would serve the purpose of verifying the modifications to the existing design curves.

With upstream transition sections identified as an important tool with which to reduce the cost of stormwater kerb inlets, further research into the application of upstream transition sections, with special reference to the relationship between the road gradient and the required length of the transition section could lead to even more cost-effective design. In the long term the research undertaken on stormwater kerb inlets needs to be incorporated in guidelines for a stormwater management programme suitable for Southern African conditions and standards as their seems to be a definite need for such a programme.

Finally, the author sincerely hopes that the results from this research project can be used by the profession in providing a better level of infrastructure to the community.

9 REFERENCES

9.1 The John Hopkins University (1956). THE DESIGN OF STORMWATER INLETS. Baltimore, Maryland. June 1956.

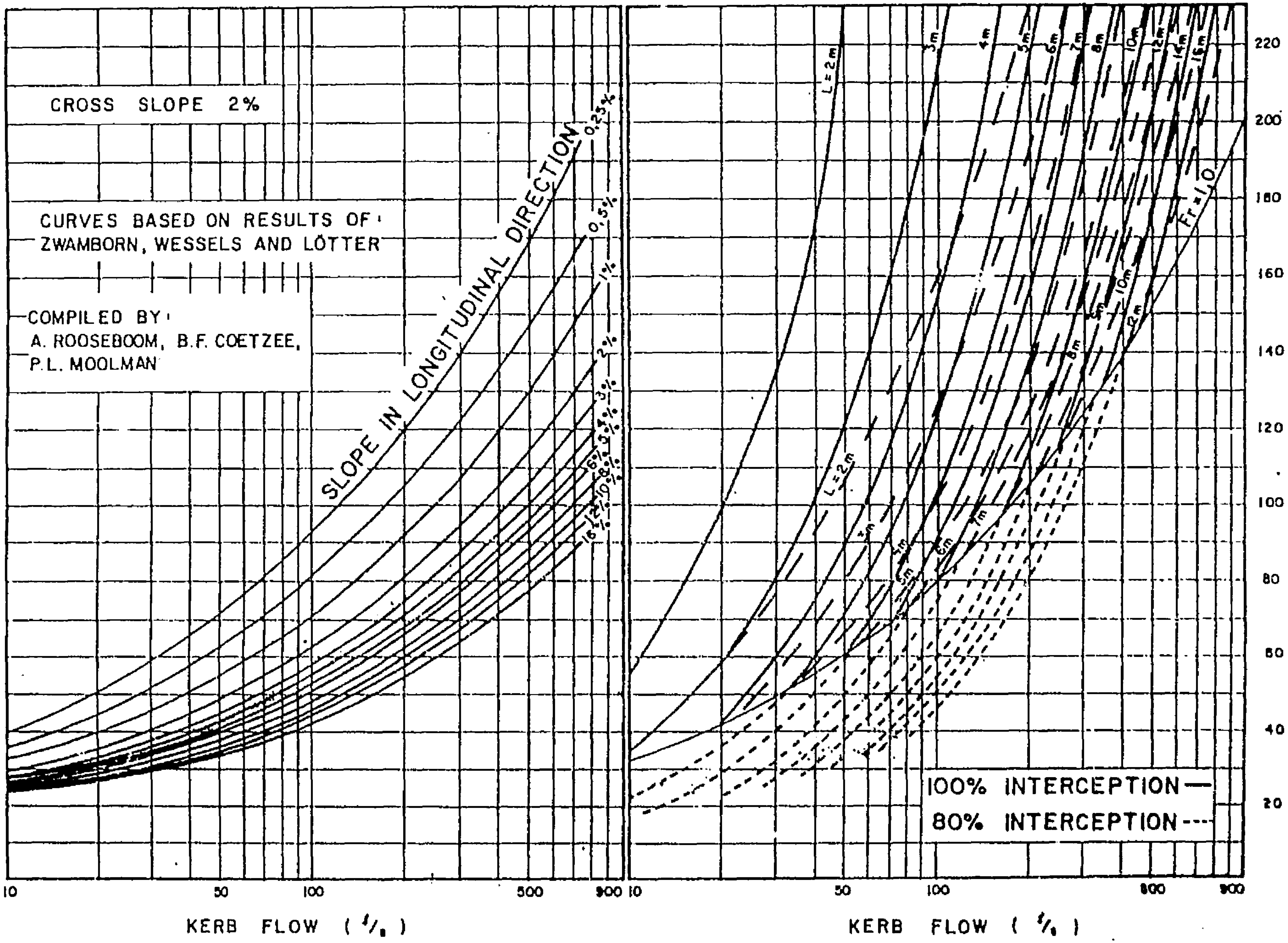
9.2 Zwamborn, J A (1966). Stormwater inlet design. PROCEEDINGS OF THE ANNUAL MUNICIPAL CONFERENCE. Johannesburg. May 1966.

9.3 Forbes, H J C (1976). Capacity of lateral stormwater inlets. THE CIVIL ENGINEER IN SOUTH AFRICA. September 1976.

9.4 Rooseboom et al (1983). ROAD DRAINAGE MANUAL. The South African Road Board, Pretoria.

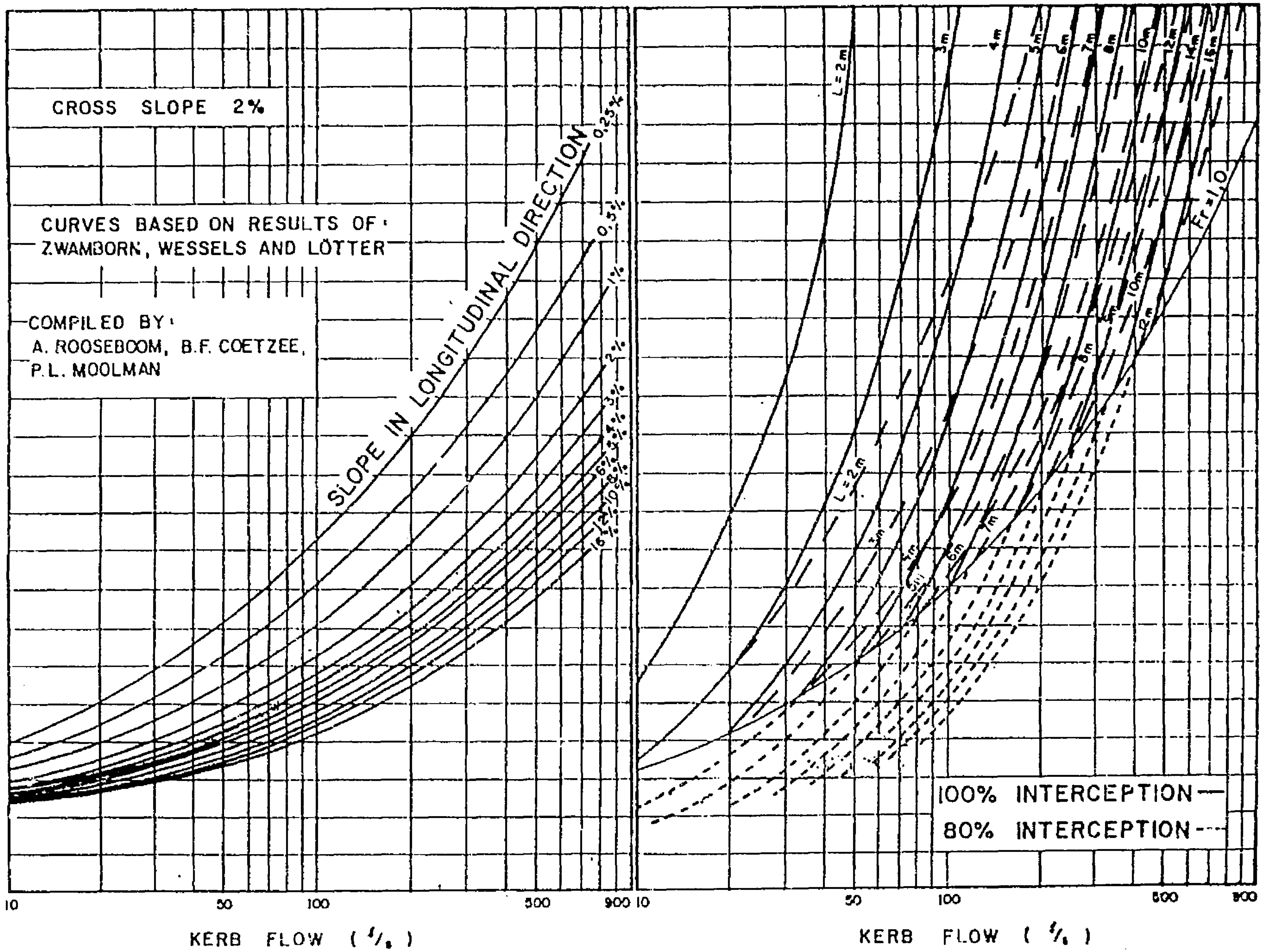
9.5 Rooseboom, A; van Schalkwyk, A; Kroon, C (1988). Modified kerb inlet design for improved hydraulic performance. THE CIVIL ENGINEER IN SOUTH AFRICA. June 1988.

9.6 Ang Alfredo H S; Tang Wilson H. PROBABILITY CONCEPTS IN ENGINEERING PLANNING AND DESIGN, Volume 1 - Basic Principles. John Wiley & Sons, Inc. New York, 1975.



ANNEXURE A

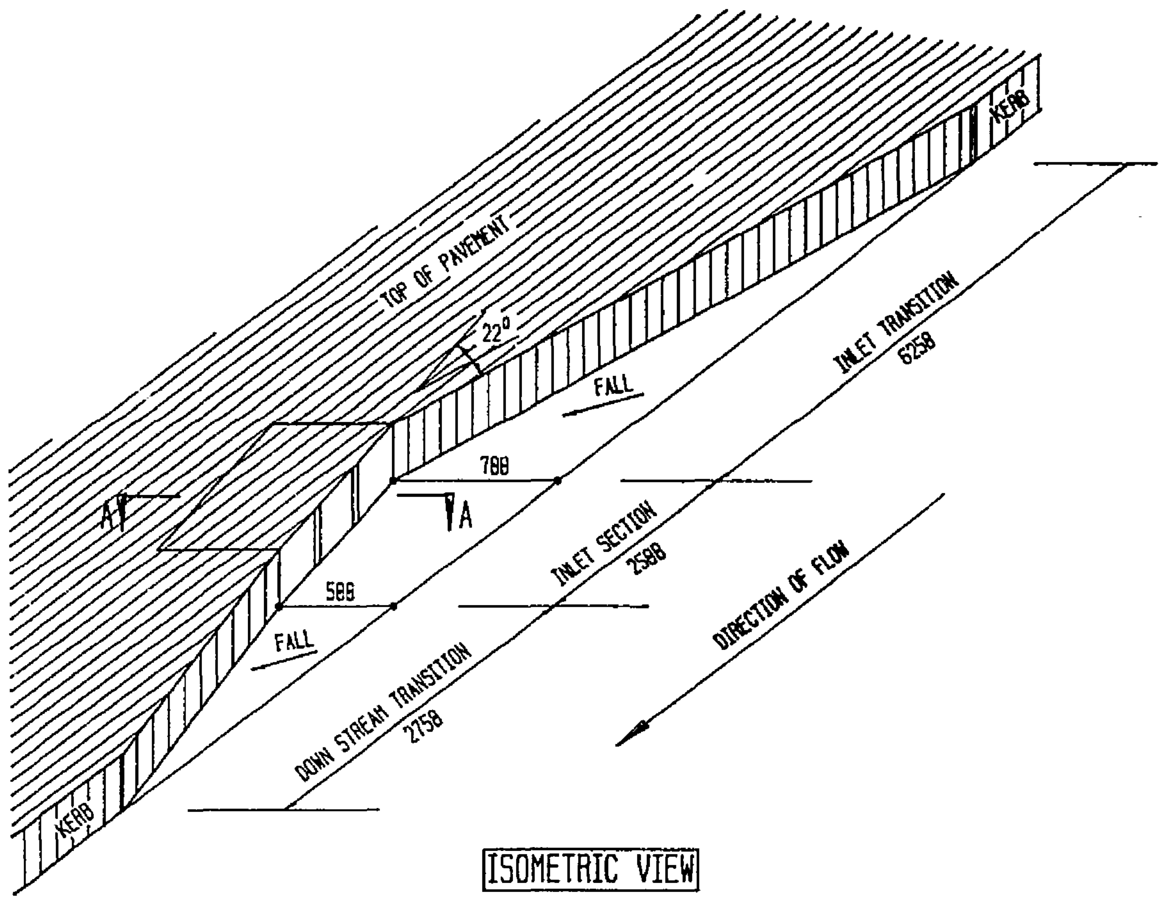
FIG. 4.3.4 (a): REQUIRED SIDE INLET LENGTHS FOR ROAD DRAINAGE: NON-DEPRESSED



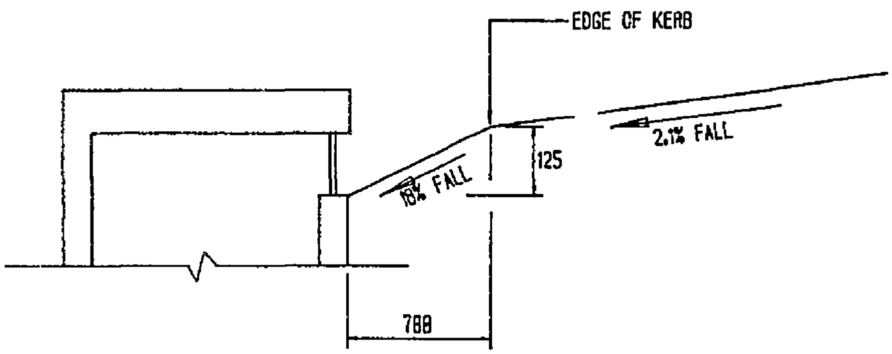
ANNEXURE A

FIG. 4.3.4 (a): REQUIRED SIDE INLET LENGTHS FOR ROAD DRAINAGE: NON-DEPRESSED

ANNEXURE B



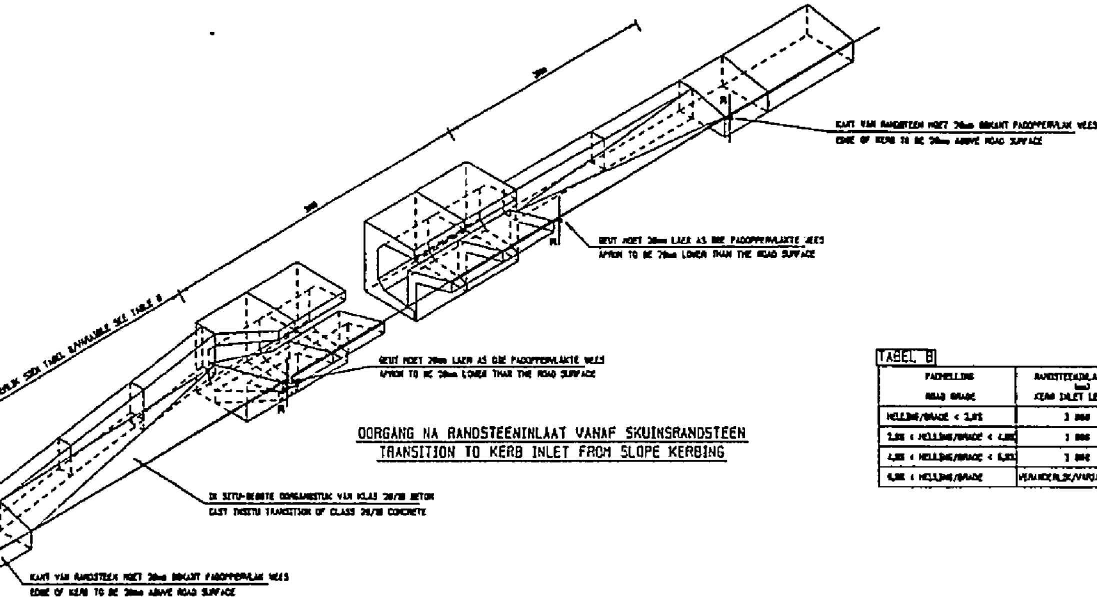
ISOMETRIC VIEW



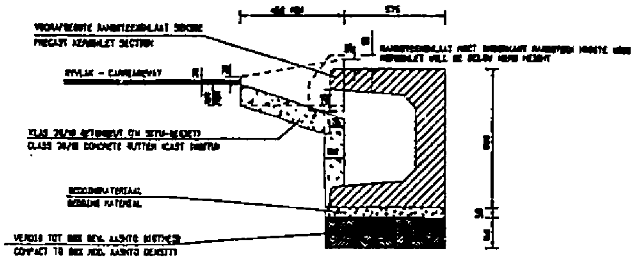
SECTION A-A

BOKSBURG-TYPE KERB INLET - TYPICAL DETAILS

ANNEXURE C

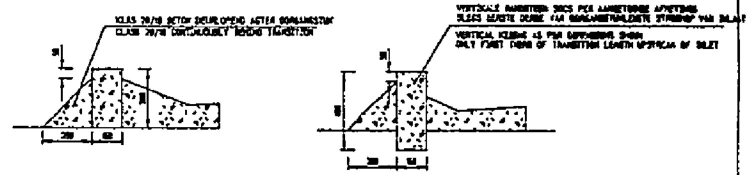


OORGANG NA RANDSTEENINLAAT VANAF SKUINSRANDSTEEN
TRANSITION TO KERB INLET FROM SLOPE KERBING



SNIT DEUR RANDSTEENINLAAT

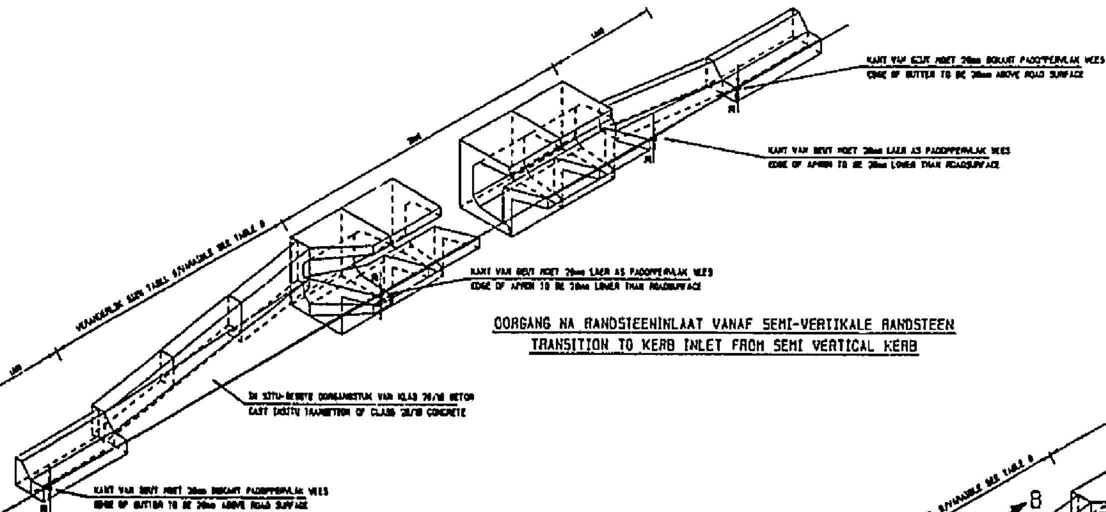
TABEL B		TABLE B
PADHELLING / ROAD GRADE	RANDSTEENINLAAT LENGTE (mm) / KERB INLET LENGTH	OORGANGSTUK LENGTE (mm) / TRANSITION LENGTH
HELLING/GRADE < 2,5%	3 000	3 000
2,5% < HELLING/GRADE < 4,5%	3 000	4 500
4,5% < HELLING/GRADE < 6,5%	3 000	5 000
6,5% < HELLING/GRADE	VERANDERLIK/VARIABLE 3 000 MIN	6 000



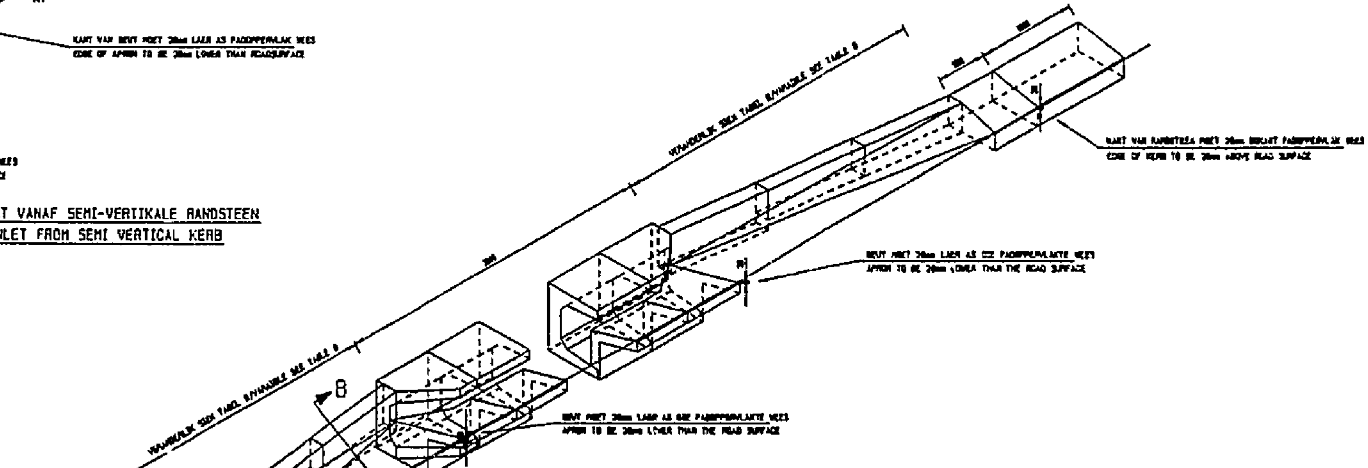
SNIT A-A
SECTION A-A

SNIT B-B
SECTION B-B

TIPIESE SNIT : RANDSTEENINLAAT OORGANGSTUKKE
TYPICAL SECTION : KERBINLET TRANSITION

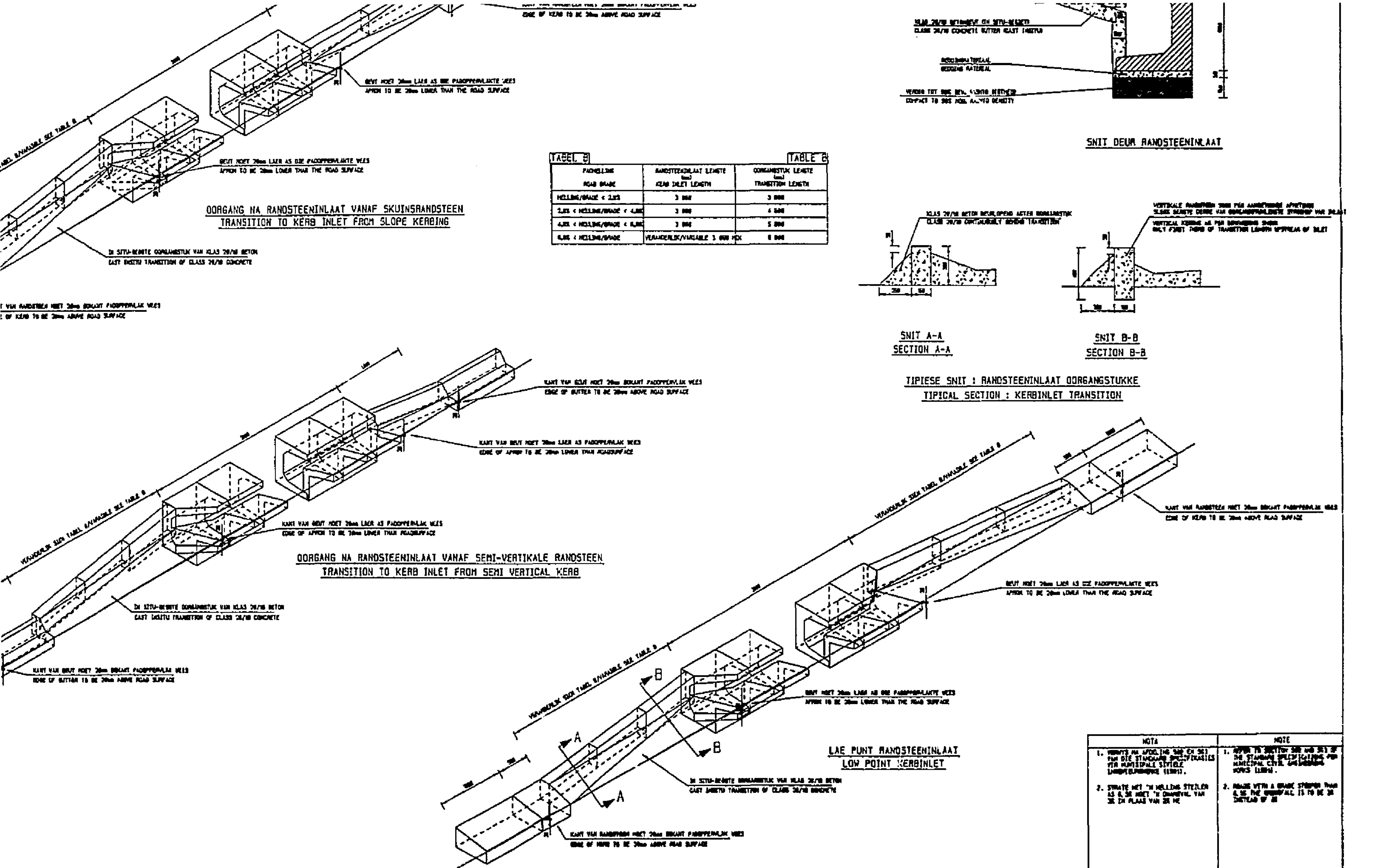


OORGANG NA RANDSTEENINLAAT VANAF SEMI-VERTIKALE RANDSTEEN
TRANSITION TO KERB INLET FROM SEMI VERTICAL KERB



LAE PUNT RANDSTEENINLAAT
LOW POINT KERBINLET

NOTA	NOTE
1. VERWYS NA AFDELING 500 EN 503 VAN DIE STANDAARD SPESIFIKASIES VIR MUNISIPALE SIVIELE INGENIEURSWERKE (1981).	1. REFER TO SECTION 500 AND 503 OF THE STANDARD SPECIFICATIONS FOR MUNICIPAL CIVIL ENGINEERING WORKS (1981).
2. STRATE MET 'N HELLING STEILER AS 6,5% MOET 'N DWARSVAL VAN 3% IN PLAAS VAN 2% HÊ	2. ROADS WITH A GRADE STEEPER THAN 6,5% THE CROSSFALL IS TO BE 3% INSTEAD OF 2%



TABEL 8 / TABLE 8

PADHELLING / ROAD GRADE	RANDSTEENINLAAT LENGTE (mm) / KERB INLET LENGTH	OORGANGSTUK LENGTE (mm) / TRANSITION LENGTH
HELLING/GRADE < 2,5%	3 000	3 000
2,5% < HELLING/GRADE < 4,0%	3 000	4 000
4,0% < HELLING/GRADE < 6,0%	3 000	5 000
6,0% < HELLING/GRADE	VERANDERLIK/VARIABLE 3 000 MIN	6 000

STADSRAAD VAN PRETORIA

CITY COUNCIL OF PRETORIA

AFDELING PAAIE

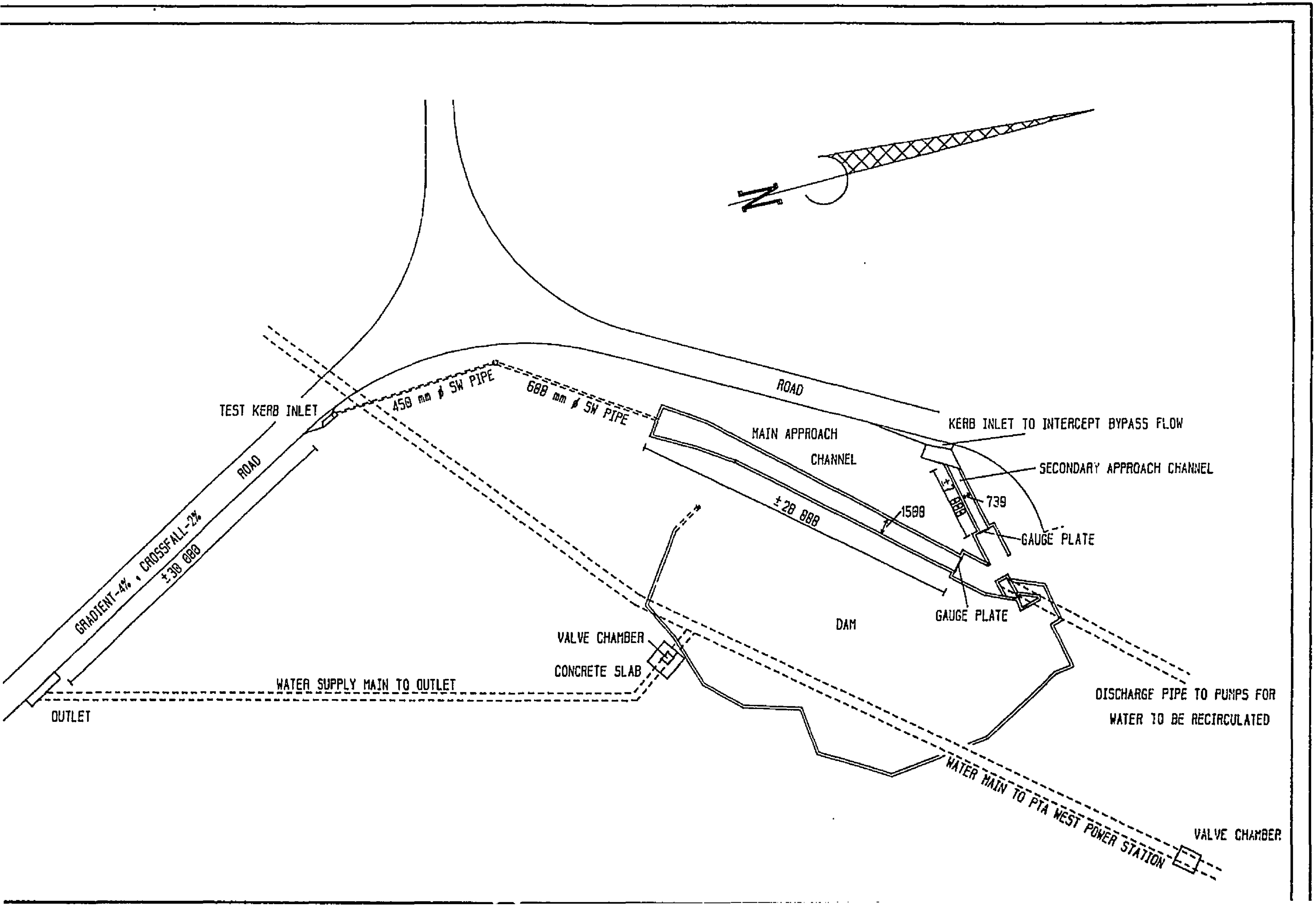
ROADS DIVISION

STORMWATERDREINERING

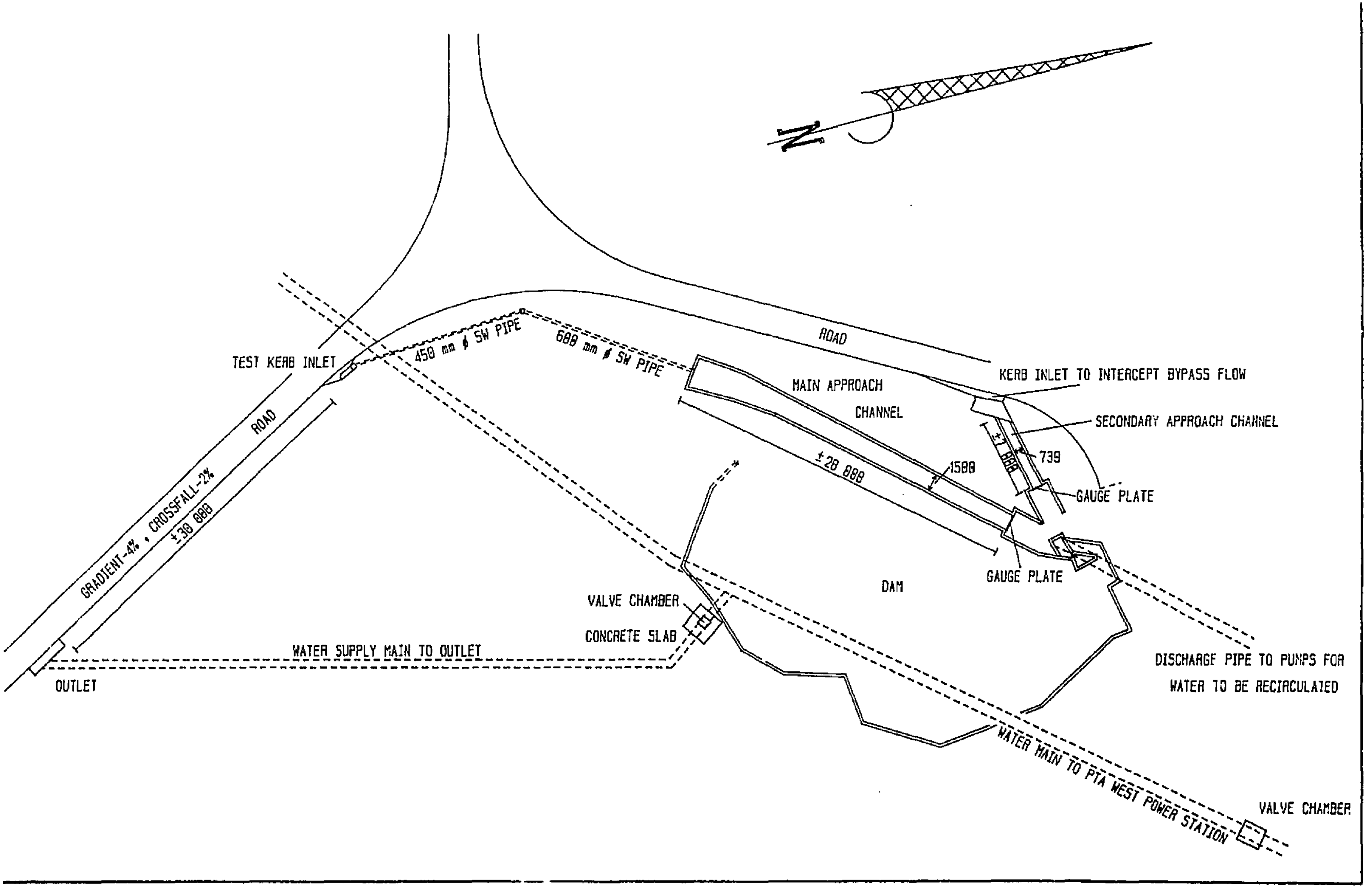
STORM WATER DRAINAGE

DETAIL VAN RANDSTEENINLAAT
DETAIL OF KERBINLET

ANNEXURE D

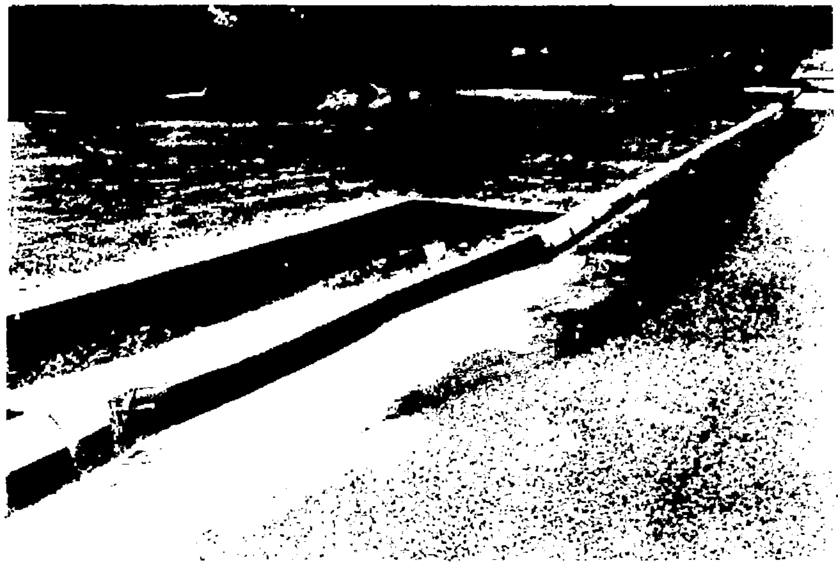


N
ROAD
TEST KERB INLET
450 mm ø SW PIPE
600 mm ø SW PIPE
MAIN APPROACH
CHANNEL
KERB INLET TO INTERCEPT BYPASS FLOW
SECONDARY APPROACH CHANNEL
739
800
1500
±20 000
GAUGE PLATE
ROAD
GRADIENT-4%, CROSSFALL-2%
±30 000
DAM
GAUGE PLATE
VALVE CHAMBER
CONCRETE SLAB
WATER SUPPLY MAIN TO OUTLET
OUTLET
DISCHARGE PIPE TO PUMPS FOR
WATER TO BE RECIRCULATED
WATER MAIN TO PTA WEST POWER STATION
VALVE CHAMBER



TEST SITE LAYOUT (NOT TO SCALE)

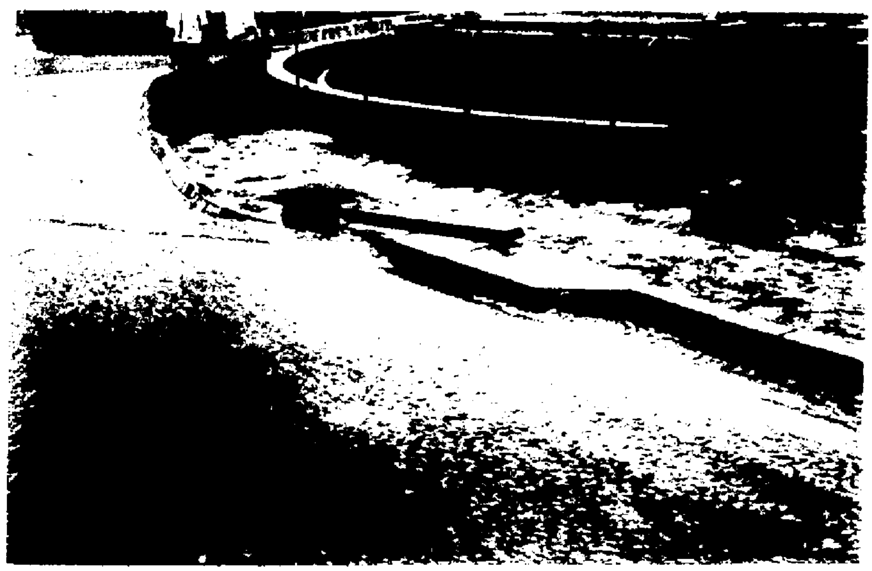
ANNEXURE E: PHOTOGRAPHS OF TEST SITE



Photograph No 1: Direction west, stormwater kerb inlet based on the conventional design i.e. without an upstream transition section. Notice the sand pit behind the kerb inlet.

Photograph No 2: Direction south, approach channels to measuring stations

Photograph No 3: Direction north west, water not intercepted by test kerb inlet flowing to the bypass kerb inlet.



Photograph No 4: Direction north west, Pretoria type kerb inlet during testing, notice the damming effect caused by the downstream transition section.

Photograph No 5: Direction south east, showing a six metre upstream transition section during testing.

ANNEXURE F1: EXPERIMENTAL RESULTS (ROAD CROSSFALL : 2.0%)

TEST NUMBER (Nr)	UPSTREAM TRANSITION LENGTH (m)	INLET LENGTH (m)	DOWNSTREAM TRANSITION LENGTH (m)	TOTAL LENGTH (m)	FLOW INTERCEPTED (l/s)	BYPASS FLOW (l/s)	TOTAL FLOW (l/s)	PERCENTAGE INTERCEPTION (%)
1.1	6	3	1	10	107.40	30.11	137.51	78.1
1.2	6	3	1	10	93.30	17.63	110.93	84.1
1.3	6	3	1	10	89.54	14.57	104.11	86.0
1.4	6	3	1	10	84.39	11.07	95.46	88.4
1.5	6	3	1	10	78.46	7.98	86.44	90.8
1.6	6	3	1	10	71.45	4.79	76.24	93.7
1.7	6	3	1	10	44.54	0.00	44.54	100.0
2.1	6	2	1	9	104.9	48.88	153.78	68.2
2.2	6	2	1	9	87.55	24.32	111.87	78.3
2.3	6	2	1	9	82.96	19.33	102.29	81.1
2.4	6	2	1	9	74.06	12.56	86.62	85.5
2.5	6	2	1	9	64.91	6.84	71.75	90.5
2.6	6	2	1	9	61.49	4.88	66.37	92.6
2.7	6	2	1	9	41.54	0.00	41.54	100.0
3.1	6	1	1	8	98.62	64.58	163.20	60.4
3.2	6	1	1	8	69.74	19.72	89.46	78.0
3.3	6	1	1	8	62.55	12.23	74.78	83.6
3.4	6	1	1	8	56.88	8.27	65.15	87.3
3.5	6	1	1	8	53.17	5.32	58.49	90.9
3.6	6	1	1	8	51.29	4.25	55.54	92.3
3.7	6	1	1	8	27.85	0.00	27.85	100.0

ANNEXURE F2: EXPERIMENTAL RESULTS (ROAD CROSSFALL : 2.0%)

TEST NUMBER (Nr)	UPSTREAM TRANSITION LENGTH (m)	INLET LENGTH (m)	DOWNSTREAM TRANSITION LENGTH (m)	TOTAL LENGTH (m)	FLOW INTERCEPTED (l/s)	BYPASS FLOW (l/s)	TOTAL FLOW (l/s)	PERCENTAGE INTERCEPTION (%)
4.1	5	1	1	7	69.25	33.70	102.95	67.3
4.2	5	1	1	7	63.73	24.10	87.83	72.6
4.3	5	1	1	7	60.44	19.57	80.01	75.5
4.4	5	1	1	7	55.07	14.13	69.20	79.6
4.5	5	1	1	7	51.84	10.42	62.26	83.3
4.6	5	1	1	7	48.90	8.16	57.06	85.7
4.7	5	1	1	7	29.83	0.00	29.83	100.0
5.1	5	2	1	8	85.05	38.38	123.43	68.9
5.2	5	2	1	8	71.82	20.37	92.19	77.9
5.3	5	2	1	8	69.98	18.49	88.47	79.1
5.4	5	2	1	8	65.15	13.73	78.88	82.6
5.5	5	2	1	8	59.52	9.91	69.43	85.7
5.6	5	2	1	8	53.52	5.29	58.81	91.0
5.7	5	2	1	8	32.90	0.00	32.90	100.0
6.1	5	3	1	9	97.25	33.40	130.65	74.4
6.2	5	3	1	9	88.74	25.02	113.76	78.0
6.3	5	3	1	9	79.61	16.03	95.64	83.2
6.4	5	3	1	9	73.93	11.11	85.04	86.9
6.5	5	3	1	9	67.79	7.09	74.88	90.5
6.6	5	3	1	9	61.84	4.10	65.94	93.8
6.7	5	3	1	9	39.32	0.00	39.32	100.0

ANNEXURE F3: EXPERIMENTAL RESULTS (ROAD CROSSFALL : 2.0%)

TEST NUMBER (Nr)	UPSTREAM TRANSITION LENGTH (m)	INLET LENGTH (m)	DOWNSTREAM TRANSITION LENGTH (m)	TOTAL LENGTH (m)	FLOW INTERCEPTED (l/s)	BYPASS FLOW (l/s)	TOTAL FLOW (l/s)	PERCENTAGE INTERCEPTION (%)
7.1	4	3	1	8	78.46	34.85	113.31	69.2
7.2	4	3	1	8	73.31	29.19	102.53	71.5
7.3	4	3	1	8	68.16	23.68	91.841	74.2
7.4	4	3	1	8	66.47	21.44	87.91	75.6
7.5	4	3	1	8	59.29	13.86	73.15	81.1
7.6	4	3	1	8	42.36	3.02	45.38	93.3
7.7	4	3	1	8	20.71	0.00	20.71	100.0
8.1	4	2	1	7	63.73	32.98	96.71	65.9
8.2	4	2	1	7	59.86	26.66	86.52	69.2
8.3	4	2	1	7	55.63	22.52	78.15	71.2
8.4	4	2	1	7	50.96	14.53	65.49	77.8
8.5	4	2	1	7	45.48	10.06	55.54	81.9
8.6	4	2	1	7	37.13	4.16	41.29	89.9
8.7	4	2	1	7	20.23	0.00	20.23	100.0
9.1	4	1	1	6	51.62	36.26	87.88	58.7
9.2	4	1	1	6	50.09	30.22	80.31	62.4
9.3	4	1	1	6	44.85	20.52	65.37	68.6
9.4	4	1	1	6	40.63	14.75	55.38	73.4
9.5	4	1	1	6	35.38	9.48	44.86	78.9
9.6	4	1	1	6	31.77	5.54	37.31	85.2
9.7	4	1	1	6	14.76	0.00	14.76	100.0

ANNEXURE F4: EXPERIMENTAL RESULTS (ROAD CROSSFALL : 2.0%)

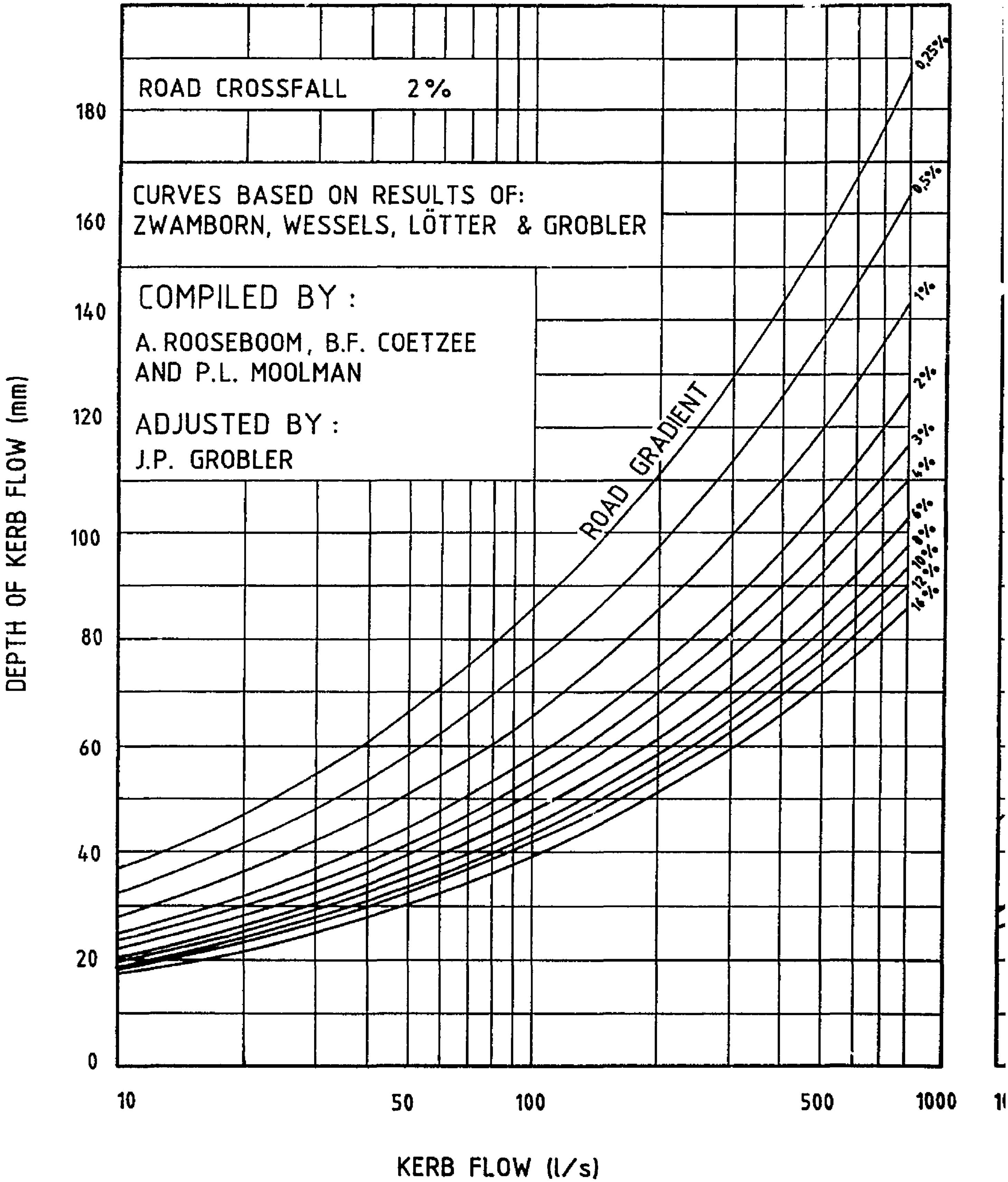
TEST NUMBER (Nr)	UPSTREAM TRANSITION LENGTH (m)	INLET LENGTH (m)	DOWNSTREAM TRANSITION LENGTH (m)	TOTAL LENGTH (m)	FLOW INTERCEPTED (l/s)	BYPASS FLOW (l/s)	TOTAL FLOW (l/s)	PERCENTAGE INTERCEPTION (%)
10.1	3	1	1	5	44.22	72.64	116.86	37.8
10.2	3	1	1	5	42.36	67.12	109.48	38.7
10.3	3	1	1	5	39.32	53.00	92.32	42.6
10.4	3	1	1	5	36.06	41.78	77.84	46.3
10.5	3	1	1	5	30.29	26.05	56.34	53.8
10.6	3	1	1	5	22.51	10.70	33.21	67.8
10.7	3	1	1	5	9.76	0.00	9.76	100.0
11.1	3	2	1	6	52.61	49.85	102.46	51.3
11.2	3	2	1	6	48.25	39.72	87.97	54.8
11.3	3	2	1	6	41.54	21.21	65.75	63.2
11.4	3	2	1	6	36.06	15.80	51.86	69.5
11.5	3	2	1	6	31.68	10.14	41.82	75.8
11.6	3	2	1	6	25.48	4.42	29.90	85.2
11.7	3	2	1	6	12.99	0.00	12.99	100.0
12.1	3	3	1	7	75.31	75.29	150.60	50.0
12.2	3	3	1	7	61.73	42.82	104.55	59.0
12.3	3	3	1	7	56.31	34.97	91.28	61.7
12.4	3	3	1	7	49.65	22.26	71.91	69.0
12.5	3	3	1	7	43.29	14.93	58.22	74.4
12.6	3	3	1	7	31.31	4.66	35.97	87.0
12.7	3	3	1	7	17.19	0.00	17.19	100.0

ANNEXURE F5: EXPERIMENTAL RESULTS (ROAD CROSSFALL : 2.9%)

TEST NUMBER (Nr)	UPSTREAM TRANSITION LENGTH (m)	INLET LENGTH (m)	DOWNSTREAM TRANSITION LENGTH (m)	TOTAL LENGTH (m)	FLOW INTERCEPTED (l/s)	BYPASS FLOW (l/s)	TOTAL FLOW (l/s)	PERCENTAGE INTERCEPTION (%)
13.1	6	3	1	10	149.16	50.19	199.35	74.8
13.2	6	3	1	10	127.99	28.90	156.89	81.6
13.3	6	3	1	10	116.97	20.32	137.29	85.2
13.4	6	3	1	10	109.15	15.25	124.40	87.7
13.5	6	3	1	10	102.79	11.48	114.27	90.0
13.6	6	3	1	10	97.38	8.91	106.29	91.6
13.7	6	3	1	10	61.96	0.00	61.96	100.0
14.1	6	2	1	9	118.70	42.17	160.87	73.8
14.2	6	2	1	9	108.00	31.27	139.27	77.5
14.3	6	2	1	9	100.00	23.94	123.94	80.7
14.4	6	2	1	9	89.00	15.84	104.84	84.9
14.5	6	2	1	9	82.18	11.35	93.53	87.9
14.6	6	2	1	9	74.81	5.80	80.61	92.8
14.7	6	2	1	9	51.40	0.00	51.40	100.0

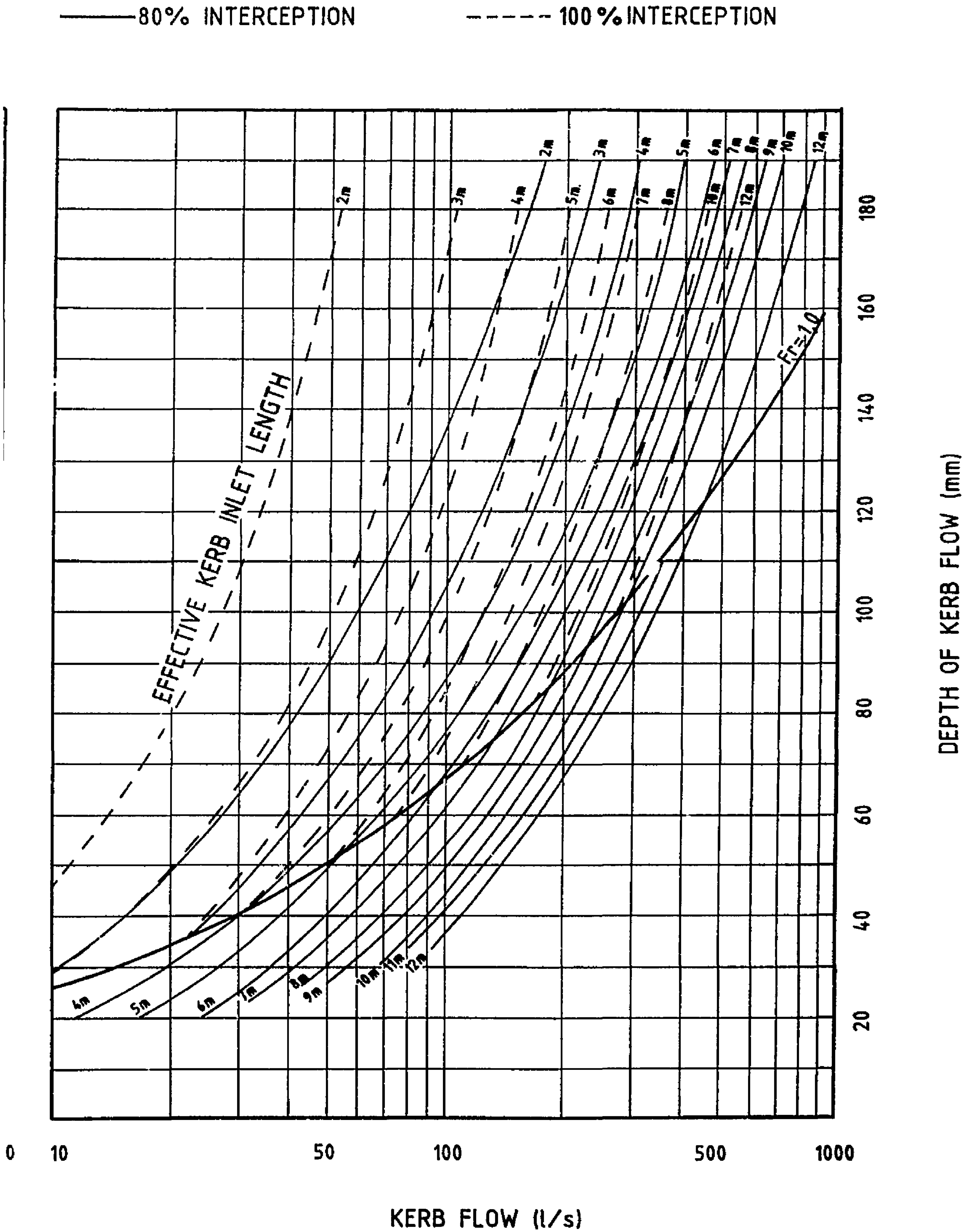
ANNEXURE F6: EXPERIMENTAL RESULTS (ROAD CROSSFALL : 2.9%)

TEST NUMBER (Nr)	UPSTREAM TRANSITION LENGTH (m)	INLET LENGTH (m)	DOWNSTREAM TRANSITION LENGTH (m)	TOTAL LENGTH (m)	FLOW INTERCEPTED (l/s)	BYPASS FLOW (l/s)	TOTAL FLOW (l/s)	PERCENTAGE INTERCEPTION (%)
15.1	4	3	1	8	78.72	30.22	108.94	72.3
15.2	4	3	1	8	76.56	27.46	104.02	73.6
15.3	4	3	1	8	70.84	21.54	92.389	75.7
15.4	4	3	1	8	66.35	16.68	83.03	79.9
15.5	4	3	1	8	61.73	10.70	72.437	85.2
15.6	4	3	1	8	49.70	3.48	53.18	93.5
15.7	4	3	1	8	32.80	0.00	32.80	100.0
16.1	4	2	1	7	79.87	55.58	135.45	59.0
16.2	4	2	1	7	69.37	37.94	107.31	64.6
16.3	4	2	1	7	62.66	28.90	91.56	68.4
16.4	4	2	1	7	57.79	21.39	79.18	73.0
16.5	4	2	1	7	49.87	12.31	62.18	80.2
16.6	4	2	1	7	40.32	4.36	44.68	90.2
16.7	4	2	1	7	21.77	0.00	21.77	100.0

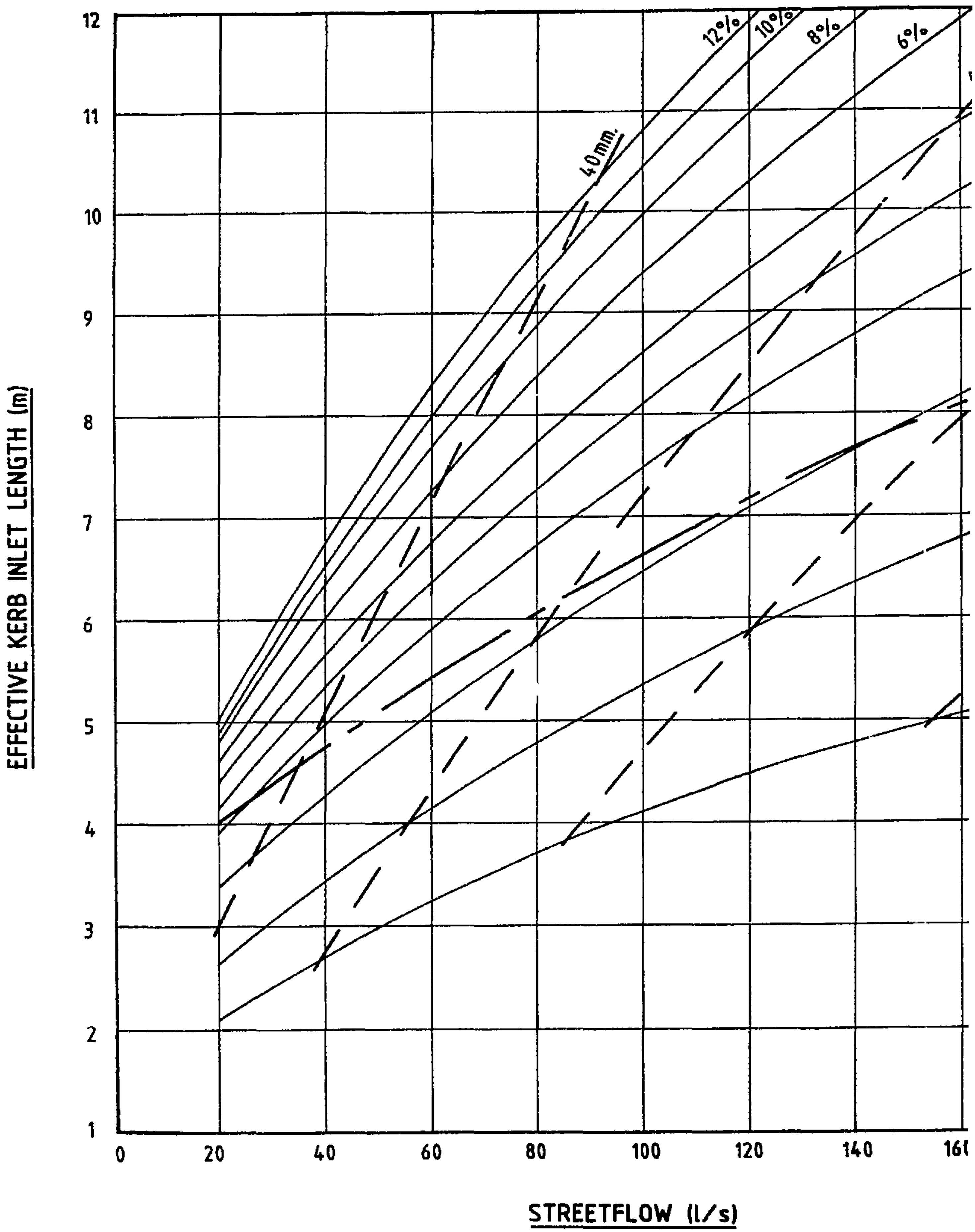


ANNEXURE A: REQUIRED SIDE INLET LENGTHS FOR

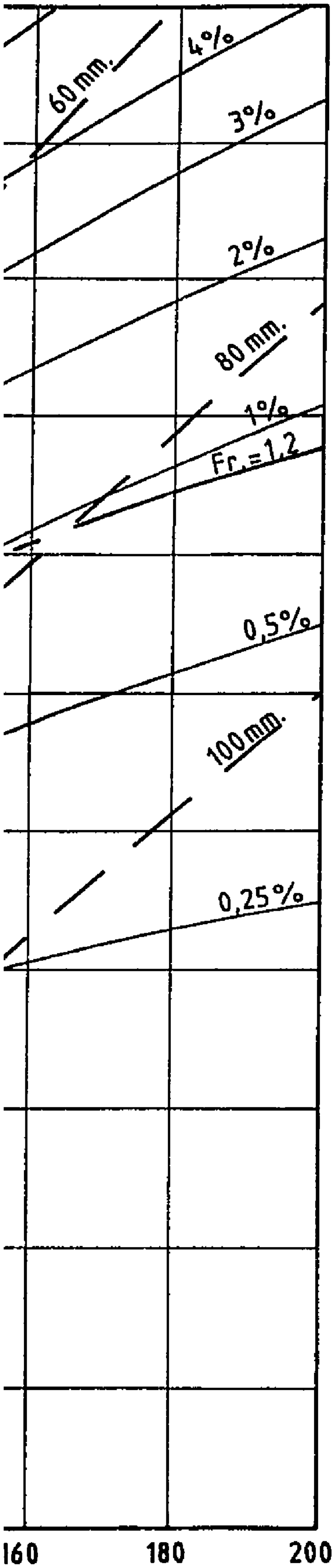
ANNEXURE G



80% INTERCEPTION
100 % INTERCEPTION
EFFECTIVE KERB INLET LENGTH
Fr=1.0
DEPTH OF KERB FLOW (mm)
KERB FLOW (l/s)
0
10
50
100
500
1000
20
40
60
80
100
120
140
160
180



NEWLY DEVELOPED DESIGN CURV



	CATEGORY		
	FROUDE NUMBER ≺ 1,2	FROUDE NUMBER ≻ 1,2 ROAD GRADIENT ≺ 3%	ROAD GRADIENT ≻ 3%
MAXIMUM RATIO (UPSTREAM TRANSITION SECTION LENGTH/ INLET SECTION LENGTH)	2	2	6
ABSOLUTE MAXIMUM LENGTH OF TRANSITION	4m	5m	6m

——————— EFFECTIVE KERB INLET LENGTH AT SPECIFIED ROAD GRADIENT

— — — — — DEPTH OF FLOW AT KERB (mm)

——— - ——— - FROUDE NUMBER = 1,2

CURVES DEPICT 80% INTERCEPTION AT SPECIFIED STREETFLOW

VES FOR STORMWATER KERB INLETS.

ANNEXURE I1: INCREASED INLET LENGTHS TO APPROACH 100% INTERCEPTION

EFFECTIVE INLET LENGTH (m)	ACTUAL INLET CAPACITIES WITH AN INCREASE OF 10% IN EFFECTIVE INLET LENGTH (FLOWS DISPLAYED IN BRACKETS INDICATE STREET FLOWS AT WHICH 80% OF FLOW IS INTERCEPTED) (l/s)						
	ROAD GRADIENT (%)						
	0.5	2.0	4.0	6.0	8.0	10.0	12.0
3.0 3.3	(28) 29	- -	- -	- -	- -	- -	- -
4.0 4.4	(55) 53	(22) 24	- -	- -	- -	- -	- -
5.0 5.5	(87) 84	(42) 42	(30) 30	(25) 25	(22) 23	(21) 21	(20) 20
6.0 6.6	(125) 122	(64) 63	(47) 46	(39) 39	(36) 35	(33) 33	(32) 31
7.0 7.7	(173) 168	(90) 87	(65) 63	(56) 54	(50) 48	(46) 45	(43) 42
8.0 8.8	- -	(118) 114	(86) 83	(74) 70	(65) 62	(60) 59	(57) 54
9.0 9.9	- -	(150) 146	(109) 106	(93) 89	(82) 78	(75) 73	(71) 68
10.0 11.0	- -	- -	(135) 132	(114) 110	(100) 96	(92) 88	(85) 83
11.0 12.1	- -	- -	- -	(137) 132	(120) 115	(110) 106	(101) 98

ANNEXURE I2: INCREASED INLET LENGTHS TO APPROACH 100% INTERCEPTION

EFFECTIVE INLET LENGTH (m)	ACTUAL INLET CAPACITIES WITH AN INCREASE OF 12% IN EFFECTIVE INLET LENGTH (FLOWS DISPLAYED IN BRACKETS INDICATE STREET FLOWS AT WHICH 80% OF FLOW IS INTERCEPTED) (l/s)						
	ROAD GRADIENT (%)						
	0.5	2.0	4.0	6.0	8.0	10.0	12.0
3.0 3.4	(28) 31	- -	- -	- -	- -	- -	- -
4.0 4.5	(55) 56	(22) 26	- -	- -	- -	- -	- -
5.0 5.6	(87) 87	(42) 44	(30) 32	(25) 26	(22) 24	(21) 22	(20) 22
6.0 6.7	(125) 126	(64) 66	(47) 47	(39) 41	(36) 36	(33) 34	(32) 32
7.0 7.8	(173) 176	(90) 90	(65) 65	(56) 56	(50) 50	(46) 46	(43) 43
8.0 9.0	- -	(118) 120	(86) 87	(74) 74	(65) 66	(60) 61	(57) 57
9.0 10.1	- -	(150) 152	(109) 110	(93) 93	(82) 82	(75) 75	(71) 71
10.0 11.2	- -	- -	(135) 136	(114) 114	(100) 99	(92) 91	(85) 86
11.0 12.3	- -	- -	- -	(137) 139	(120) 120	(110) 110	(101) 102

ANNEXURE J: STATISTICAL ANALYSIS OF SAMPLES

The experimental data obtained from the full scale model tests represented a small sample of a population with an unknown distribution and variance. This meant that an exact confidence interval for the population mean could be determined provided that the underlying population is normally distributed. The t-distribution (Student's t-distribution) with (n-1) degrees of freedom (where n is the sample size) is best suited for this type of application (Ang & Tang, 1975). The t-distribution has a bell-shape density function similar to the normal curve and is symmetrical about the origin. For small samples and therefore small degrees of freedom the t-distribution is flatter than the standard normal distribution, however as the sample size increases it tends toward the standard normal distribution. On this basis, the following probability statement can be formed.

$$P\,(-t_{\alpha/2,n-1} < (X-\mu)\,/\,S/n^{0.5} \leq t_{\alpha/2,n-1}) \qquad = 1-\alpha \qquad \text{(J.1)}$$

$t_{\alpha/2,n-1}$: value of variate T at the cumulative probability $1-\alpha$ for n-1 degrees of freedom

$1-\alpha$: specified confidence level

n : sample size

n-1 : degrees of freedom

X : sample mean

μ : population mean

S : sample standard deviation ($=\sigma$: population standard deviation, assumed)

If Equation J.1 is rearranged the exact confidence interval for the mean is:

$$<\mu>_{1-\alpha} = [\ x - t_{\alpha/2,n-1} .\ S/n^{0.5}\ ;\ x + t_{\alpha/2,n-1} .\ S/n^{0.5}\ ] \qquad (J.2)$$

x : sample mean

$<\mu>_{1-\alpha}$: confidence interval for specified confidence level of $1-\alpha$

If the population is not normally distributed Equation J.2 may still be used to determine the approximate confidence interval for the sample mean if the sample size is in the region of n=10 or more.

In Chapter 5 "erroneous" experimental results obtained from tests on a kerb inlet with an upstream transition section of 3 metres were identified. The extent of the construction fault resulting in the "erroneous" test results was discussed in the above-mentioned chapter.

Analysis of the sample of test results with and without the "erroneous" test results lead to the determination of confidence intervals for a 95% confidence level that overlapped to a large extent. The calculation of the confidence intervals is shown below.

Confidence interval for μ including "erroneous" test results

The values are : -21, 6, 50, -11, 44, 67, 35, 54, 53, 46, 47, 39

n : 12

n-1 : 11

$1-\alpha$: 0.95

$\alpha/2$: 0.025

$t_{0.025,11}$: 2.201

x : 34.08

s : 27.60

$$<\mu>_{0.95} = [34.08 - 2.201.(27.60/12^{0.5}) ; 34.08 + 2.201.(27.60/12^{0.5})]$$
$$= [16.54 ; 51.62]$$

Confidence interval for μ excluding "erroneous" test results

The values are	:	50, 44, 67, 35, 54, 53, 46, 47, 39
n	:	9
n-1	:	11
$1-\alpha$	:	0.95
$\alpha/2$	:	0.025
$t_{0.025,8}$	:	2.306
x	:	48.33
s	:	9.33

$$<\mu>_{0.95} = [48.33 - 2.306. (9.33/9^{0.5}) ; 48.33 + 2.306. (9.33/9^{0.5})]$$
$$= [41.16 ; 55.50]$$